University of Alberta

Evaluation of Tunnel Lining Systems for Internal and External Pressure

By

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Evaluation of tunnel lining systems for internal and external pressure

Abstract

The City of Edmonton has constructed over 2000 km of sewer tunnels since the 1950s, and sewer tunnels will continue to be constructed in the future to improve the water quality of the North Saskatchewan River and protect the city's properties from flooding.

The majority of sewer tunnels in Edmonton are gravity flow; however, the W12 project's tunnel will be pressurized as an inverted siphon tunnel in order to cross under the river, and may be regarded as unique in that the internal pressure exceeds external pressure. Typical tunnel lining systems are comprised of cast-in-place concrete or unbolted pre-cast concrete segments, and these systems generally support only overburden pressures.

This study presents a framework for evaluating the three types of tunnel lining systems for internal and external pressure using performance indicators. The proposed framework in this study involves a systematic review of the technical basis supporting the primary decisions made with regard to pressurized tunnel design.

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CHAPTER 1

INTRODUCTION

1.1 Introduction

The City of Edmonton has determined that between 60 and 80% of the City's Combined Sewer Overflows (CSOs) occur at the Rat Creek CSO located across from the Riverside Golf Course, just downstream of the Dawson Bridge in Edmonton. CSOs during wet weather events can be significant sources of pollutants in the North Saskatchewan River.

In fact, Alberta Environment has requested that the City of Edmonton move towards either eliminating CSOs or undertaking improvements that will present comparable environmental benefits. This request has led to the development of several long term strategies for improvements to the City's sewage conveyance and treatment systems, known as the "CSO Control Strategy", to reduce the impacts of CSOs. The W12 project is one of the main strategies formulated to reduce pollution into the North Saskatchewan River, and will result in a significant environmental improvement through the reduction of CSOs. The W12 project, an inverted syphon tunnel at Rat Creek, will assist in reducing the volume of CSOs by as much as 56-86% and in decreasing the number of annual CSO occurrences from 89 to 46.

The W12 project can be divided into two subprojects according to the tunneling method used; one is excavated by an open-faced tunnel boring machine that is equipped to erect steel ribs and lagging for the primary liner, and the other is excavated by using an Earth Pressure Balance Machine (EPBM). Tunneling conditions in this area are similar to those associated with recent bedrock tunnels, and the equipment required for this work is feasible. However, the tunneling requirements should only be evaluated after a detailed investigation. The tunnel lining systems used in similar tunneling projects in Edmonton provide temporary support of ribs and lagging, and incorporate either cast-in-place concrete or one-pass segmental concrete as a permanent liner. Unfortunately, these lining systems have never been used in Edmonton in tunnels deeper than about 66m, and never with the internal pressure that will be required for this tunnel.

A cast-in-place concrete permanent liner with steel ribs and timber lagging may provide greater flexibility in the event that disturbed geological conditions, (i.e. abandoned mines), are encountered. The reinforced concrete liner is designed to withstand the internal hydraulic pressure and overburden pressures in tunnels. The single pass unbolted pre-cast segmental concrete liner, used in recent tunneling projects in Edmonton, is not recommended for this tunnel. A key feature of this tunnel is that the internal pressure exceeds the overburden pressure and so there is potential for a hydraulic fracture of the ground resulting in liner separation in the case that an unbolted segmental liner is used. It is possible, however, to use cast-in-place reinforced concrete, a bolted and gasketed pre-cast segmental concrete, or a pre-cast pipe as a permanent liner.

This study presents a framework for evaluating three types of tunnel lining systems, taking into account durability and water-tightness using simple performance indicators. The absolute value of durability and leakage prevention may differ between the various tunnel lining systems. However, the relative significance of the variable evaluation factors involved is considerable, and these factors can be adequately evaluated by surveying experts' opinions. This study involves a systematic review of the technical basis behind the critical decisions made with respect to pressurized tunnel design. These evaluations and the literature review lead to the designation of steel or polypropylene fiber reinforced concrete as the most feasible means of improving durability and reducing leakage. This paper also offers a mixing design for the proposed tunnel lining system.

1.2 Research objective

The City of Edmonton has a combined total of approximately 4 600 km of sewer pipes, and over 2 000 km of these sewer pipes have been constructed by means of a tunneling method. In addition, planning is underway for approximately 20 km of new sewer tunnels to be constructed at depths of up to 80 m. Rapid economic and infrastructural expansion of the City has resulted in an increase in tunneling projects over the last several years. The City began developing its tunneling expertise with hand tunneling in the 1950s, and since then the City has in its possession several Tunnel

Boring Machines (TBMs). The City purchased a new Earth Pressure Balance Machine (EPBM) in 2006 specifically for the W12 project. Moreover, the W12 project will incorporate excavation by each of these two types of machinery, depending on the geological conditions.

The existing sewer tunnels are typically about 2.3 m in diameter, and the tunnel lining systems generally consist of pre-cast segmental concrete liner and cast-in-place concrete with steel ribs and timber lagging. According to the City of Edmonton, these lining systems have never been used in depths exceeding 66 m or in environments with relatively high internal pressure. This study will focus on the following:

- Use of performance indicators, to evaluate three types of tunnel lining systems and to recommend the most appropriate tunnel lining system for W12 project
- Finding the most important factor to affect the performance of tunnel lining system
- Assessing the feasibility of using steel fiber reinforced concrete to enhance the durability of tunnel lining based on literature review

1.3 Outline of thesis

Chapter 1 includes an introduction to the issues involved in evaluating tunnel lining systems and the rationale for examining the W12 project in this study. Chapter 2 is a review of the literature pertaining to tunnel construction and lining systems, including the general tunneling process, open TBMs, and EPBMs. The steel fiber-reinforced concrete lining system will also be discussed in this section. Chapter 3 accounts for the typical characteristics of tunnel lining systems used in Edmonton, such as cast-in-place concrete and pre-cast segmental concrete. Chapter 4 discusses performance indicators used for the evaluation of tunnel lining systems in relation to internal and external pressures, and analyzes the results of the evaluation. The feasibility of steel fiber reinforced concrete as an alternative lining system is explored by means of existing data and literature. These include laboratory tests and field applications. This chapter also describes improved concrete pumping and placement methods for cast-in-place reinforced concrete when steel fiber is used. Finally, chapter 5 concludes the evaluation and offers recommendations for future projects.

Chapter 2

Literature review and current practice

2.1 Introduction

One of the main tasks at the design stage of tunneling projects is to evaluate and select a suitable tunnel lining system and excavating machine. This requires a considerable amount of experience and both practical and theoretical knowledge. In the case of the W12 project, two types of machinery have been selected, but a permanent lining system has not yet been selected for either section. In reality, a review of previous and similar projects is also informative for both design and construction. In this chapter, a literature review will focus on several important milestones in tunnel design and equipment selection, referring both to basic theory and real-life applications. As an alternative lining, steel fiber reinforcement concrete will be reviewed.

2.2 Tunnel lining systems

The design of tunnel liners should satisfy structural and operational functions. In terms of its structural function, the most critical requirements are that the liner support external and internal loadings and pressures for design life and effectively control the leakage of groundwater. The operational function is to sustain an internal pressure and to ensure the appropriate operation of the tunnel.

The tunnel lining system is selected only after considering a number of factors, such as characteristics of the facilities involved, geological conditions, analysis of ground-lining interaction, and construction method. Furthermore, the installation of lining systems must not be considered as an independent phase of the tunneling process. Excavation method, for example, which must accommodate particular ground movements, will be a key determinant of which lining system is chosen. Constructability, time, and cost are also important factors affecting lining design that emphasize practical and economical issues. After construction, long service life of a tunnel is an utmost priority, and this factor is tied directly to the durability of the concrete lining.

The International Tunneling Association suggests a tunnel lining design procedure following the steps outlined in figure 2-1. First, the dimension of the tunnel is defined on the basis of operation and construction constraints. Next, one must determine which loading conditions are acting on the lining: including overburden pressure, internal and external water pressure, thrust pressure of tunneling machine, etc. Third, the lining should be defined in terms of strength and other such material properties. Finally, one must calculate member forces and check that the lining meets acceptable safety standards. Figure 2-1 shows the flow chart for the shield tunnel lining design procedure.

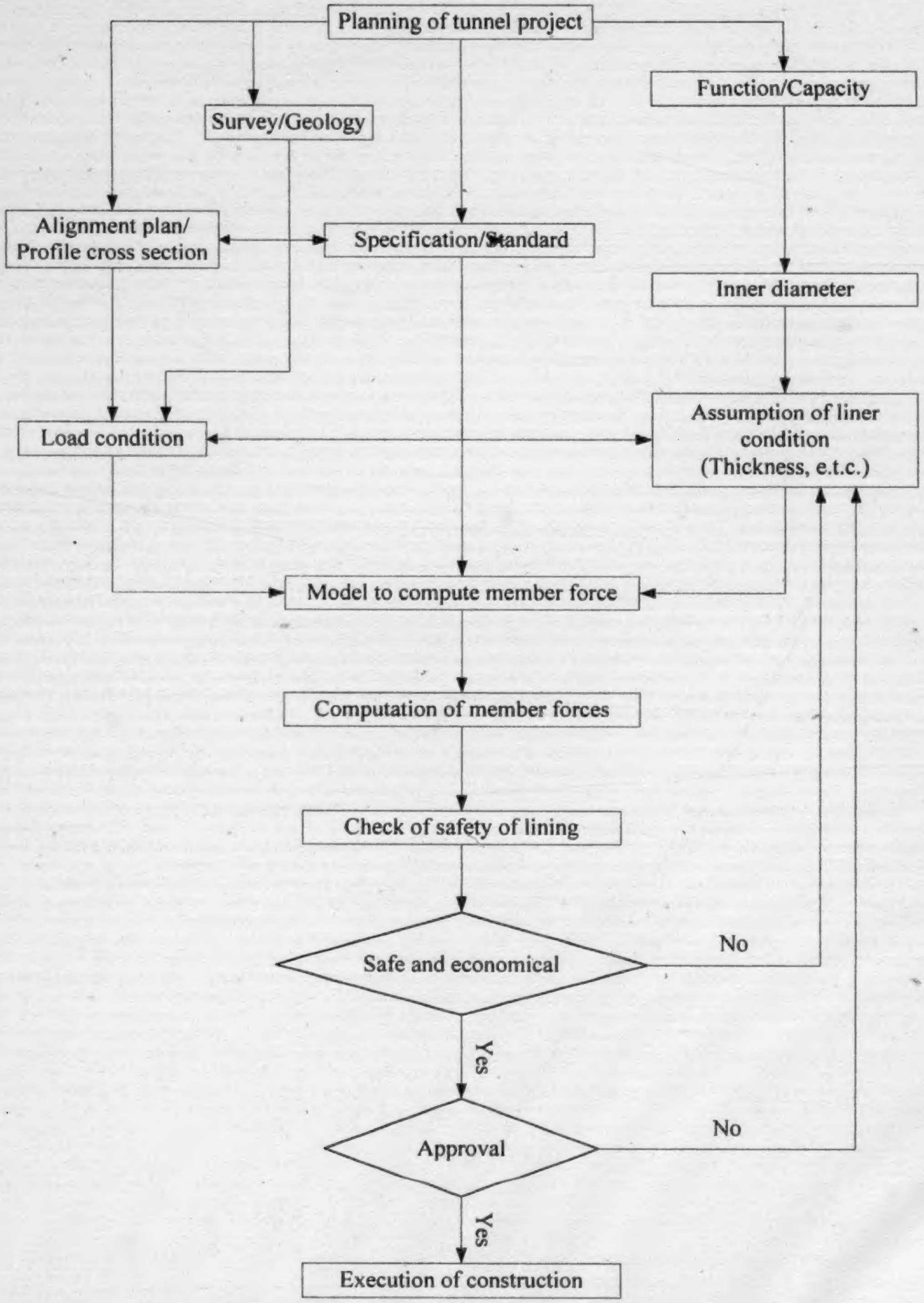

Planning of tunnel project
Survey/Geology
Function/Capacity
Alignment plan/ Profile cross section
Specification/Standard
Inner diameter
Load condition
Assumption of liner condition (Thickness, e.t.c.)
Model to compute member force
Computation of member forces
Check of safety of lining
Safe and economical
No
Yes
Approval
No
Yes
Execution of construction

Figure 2.1 Flow chart of shield tunnel lining design. (International tunneling association)

2.2.1 Ground load on the linings

The determination of ground load on the lining is an important dimension of the design stage. Mair and Taylor (1997) research the effect of ground load on tunnel lining and several of these measurements show a rapid build-up of ground loading, within a few weeks to a year, to a maximum value of overburden pressure. The measurement indicates that the vertical loads shortly after construction were equivalent to about 30% of the total overburden pressure, and that they had steadily increased to about 60% of the overburden pressure and appeared to have almost stabilized. These results comply well with the field measurement of tunnel lining load in SW1 for 3 years after lining installation (Bobey et al, 2004). The horizontal load was about 70% of the vertical load, despite the fact that the Ko (the coefficient of horizontal earth pressure at rest) would have been rated at 1.5-2 prior to tunnel construction, with the London clay being highly consolidated.

The soil displacements that occur prior to installation of the lining clearly have a major influence in reducing both the short-term and long-term ground loading to much lower values than the original in situ stress (Mair and Taylor, 1997). Particularly in the case of highly over-consolidated clays, for which the Ko-value is usually considerably greater than 1, it is usually erroneous to consider the tunnel lining being subjected to higher horizontal than vertical ground loading. According to this research, the annual lining loads vary from about 40-60% of the overburden pressure. Time predictions of the development of lining loads in clay soils is very complex, and requires a knowledge of the drainage boundary conditions as well as variations in ground permeability at varying distances from the tunnel.

With respect sand and gravel, it should be noted that the ground loading characteristics and deformations of a tunnel in dense and gravelly soils are much smaller than in clays and silts, provided any adverse water conditions are dealt with effectively during excavation (Mair and Taylor, 1997). For deeper tunnels below the water table, the majority of ground loading is the result of water pressure, with the effective stress component being very low.

Kim et al (1998) and Bobey et al (2004) show the results of lining loads measurements for several tunnels in Edmonton. Table 2-1 shows the existing data. These field measurements cover a monitoring period extending about 3 years after liner installation. The instrumented lining load pressures are expressed in terms of the factor "n", which is the vertical lining pressure (Pv) divided by the product of material bulk unit weight (Ws) and tunnel diameter (D). The lining pressure may also be expressed as a percentage of overburden pressures. Results indicate that the lining pressure over the monitoring period range from 8-87% of the full overburden pressure. It is also worth noting that the lining loads installed in bedrock were all less than 50% of the full overburden.

Table 2-1. Summary of field measurement of tunnel lining load (Bobey et al, 2004)

Tunnel	Rock or soil	Diameter (m)	Depth (m)	Measured lining pressure factor (n)	% of full overburden pressure
E.L. Smith	Clay shale	2.5	66	0.7-3.0	8-36
Whitemud Creek	Clay shale	6.1	47	1.1	14
Highway 16	Rafted bedrock & till	2.6	3.8 to 4.5	0.9-1.5	50-87
SESS SW1	Clay shale	2.3	45	3.2	20

2.2.2 Risk in tunneling

In the case of underground construction, comprehensive and realistic plans are required in order to minimize time spent, cost, and risk. One of the most important tasks involved in tunnel design is to decide on a suitable excavation method and supporting systems according to the tunnel profile. Uncertainty about the ground conditions is one of the main causes of delay and cost overruns in tunneling construction. The gap between geotechnical reports and the actual ground conditions to be encountered necessitates an extensive effort to manage and reduce risk. In geotechnical construction, it is common to find sources of uncertainty related to the following factors (Flores, 2006):

- Spatial variation and scale effect (heterogeneity of the soil mass)
- Limited soil investigation (insufficiently defined parameters)

- Lack of agreement between field and laboratory test
- Measurement errors (lack of precision of instruments)
- Subjective estimation
- Random nature of static and dynamic loading
- Environmental conditions (water pressure, erosion, water table fluctuations, etc.)
- Validity and accuracy of geo-mechanical models
- Use of empirical correlation
- Human error

It is common to observe discrepancies between theoretical predictions and the actual underground conditions, with a number of different variables obscuring the analysis. The potential risks can be divided into three basic groups (Munich Re group, 2004):

- Material damage to the construction work, machinery, plant and equipment
- Material damage to third-party property and resulting liability claims
- Bodily injury to employees or third parties

The selection of tunneling method for a project depends primarily on the anticipated geological conditions of the tunnel, which are the aggregation of states of important rock mass properties such as rock type and discontinuity (Likhitruangsilp and Ioannou, 2004). The selected tunneling method should be adjustable to expected underground conditions without seriously interrupting the excavation process. Geological uncertainties can also affect the productivity of the tunneling processes, as they may give rise to variations in construction equipment performance and unexpected accidents during construction.

2.2.3 Pressurized tunnel lining

Two important purposes of concrete tunnel lining are to ensure safety of structure and to prevent leakage. Furthermore, cracking in the concrete lining may be attributed to a high internal pressure and cracks may lead to main reason of water leakage. Chen W. N. (1998) investigated existing concrete crack formulas, proposed a crack formula for immature concrete in pressure tunnel designs, and suggested critical reinforcing ratios for crack control in concrete lining as in table 2-2. This work suggested 0.2 mm as a practical crack width for design, and to optimize leakage control of pressure tunnels. This value is more conservative than that of the Design and Construction Standards (City of Edmonton, 2004), which designated a crack width value of 0.3 mm.

Table 2-2 Critical reinforcing ratios (Chen, W. N., 2004)

Concrete compressive strength of 28days, in psi	The concrete direct tensile strength of 3days, in psi	Critical reinforcing ratio
3,000	160	0.0027
4,000	190	0.0032
5,000	210	0.0035

However, this critical reinforcing ratio is merely derived from a series of crack formulas, so laboratory testing will be required to justify the above results. Chen's (1998) critical reinforcing ratio is greater (i.e. more conservative) than the minimum percentage, which is between 0.18 and 0.20%, of ACI criteria (ACI 224R, 2005).

Gabriel Fernandez (1994) evaluated the water-tightness of non-lined and lined tunnels based on an estimation of the leakage and the pore water pressures induced in the surrounding rock mass. Where the rock mass is relatively permeable, (a rock mass permeability in excess of $1*10^{-5}$ cm/sec), the desirable liner system can be chosen based on the circumferential strain level generated by the internal tunnel pressure. A series of design guidelines are summarized as follows:

(1) If the magnitude of the strain induced in the liner is lower than $1.5*10^{-4}$, a non-reinforced concrete ring with a completely contact grouting could provide a relatively "impermeable" barrier with an average permeability in the range of 10^{-7}-10^{-8} cm/sec.

(2) If the magnitude of the tensile, circumferential strain in the liner exceeds $1.5*10^{-4}$, longitudinal cracks will develop in the concrete, thus increasing the permeability of

the liner. Steel reinforcement can be used to control the width and spacing of longitudinal cracks, maintaining a permeability low enough that the liner behaves as an effective flow barrier. If the circumferential strain induced in the liner is lower than $4.0*10^{-4}$ and the rock-mass permeability is larger than 10^{-5} cm/sec, then the use of a plain concrete liner with a consolidation grouting program can be considered.

(3) If the strain level exceeds $4*10^{-4}$, large tension cracks will develop across the non-reinforced liner and extend into the adjacent, grouted rock mass, substantially increasing the permeability of the liner-grouted rock system. A reinforced concrete liner can be used to reduce the strain level within the grouted zone around the liner, maintaining its low permeability. The amount of reinforcement installed should be sufficient to maintain a strain in the liner below $6*10^{-4}$ to preclude the propagation of tension cracks across the grouted rock mass, and to maintain a permeability compatible with the grouted rock mass.

(4) If the estimated circumferential strain in a well-reinforced concrete liner exceeds the $8*10^{-4}$ value, a thin steel membrane embedded within the concrete liner can be considered. Design criteria require the circumferential strain in the steel to be maintained below $1*10^{-3}$.

2.3 Steel fiber reinforcement concrete for tunnel lining

2.3.1 Cracks and permeability

In a pressurized tunnel, water leakage resulting from cracks and permeability is one of the most important factors to take into account when considering the long-term durability of tunnel lining. Mashimo et al. (2006) investigated the effect of fiber-reinforced concrete on lining cracking through a series of laboratory and on-site experiments. This investigation demonstrated and compared the different mechanical attributes of cracking in steel fiber-reinforced concrete and plain concrete in tunnel liners. According to this study, the crack occurrence of SFRC was faster than that of plain concrete under the same curing and environmental conditions. However, most of the recorded crack widths in the section with plain concrete were measured to be approximately 0.5mm, while the crack widths with SFRC *ranged* from 0.2 to 0.5mm. It is by virtue of this fact that the lining with SFRC has a tendency to show cracks sooner, but with less development of the crack width. One can also anticipate that the long-term durability of concrete with SFRC may be improved.

Rapoport et al. (2002) investigated and tested the relationship between permeability and crack width in cracked steel fiber-reinforced concrete. This research showed that for larger crack widths, steel reinforcing macro-fibers reduce the permeability of cracked concrete. The higher steel volume of 1% reduces the permeability more than the lower steel volume of 0.5%, which nevertheless has lower permeability than that of non-reinforced concrete. This trend can be anticipated by virtue of the crack bridging associated with steel fibers, as well as the resulting multiple cracks associated with steel fiber reinforcement. For cracks smaller than 100 microns, steel reinforcing macro fibers do not seem to affect the permeability of the concrete. This threshold would still exist for the fiber reinforcement concrete because the steel fibers do not alter material porosity. In addition, the steel fiber reinforcement augments the crack geometry from one large crack to multiple, smaller cracks because the steel fibers distribute the stress evenly throughout the material. In other words, because permeability is related to the crack width, several smaller cracks will be less permeable than one large crack. Thus, although the optimum fiber volume is closely related to material porosity, it is possible to achieve a higher volume of steel fiber which will better reduce the actual permeability of crack concrete.

Banthia and Bhargava (2007) examined the permeability of unstressed concrete and evaluated the effect of fiber reinforcement. They used virgin, fully purified plantation softwood fibers with a specific gravity of 1.1, a tensile strength of 750 MPa, and an elastic modulus of 8.3 Gpa, and with an average length of 2.3 mm. The results of permeability tests showed that a reduction in the water permeability of unstressed concrete due to fiber reinforcement is in agreement with the results of Rapoport et al (2002). They proposed that a reduction in permeability due to fiber reinforcement can be related to two known mechanisms. First, fibers produce mixture stiffening, reduce the settlement of aggregates, and decrease bleeding. This may serve to reduce the formation of bleed channels and decrease the ease with which flow can occur through the material. Second, hydrophilic fibers such as cellulose are likely to better engage water in the mixture and decrease overall early-age shrinkage. The apparent ability of a fiber to reduce the permeability of unstressed concrete can be affected by the mixture design, fiber type, volume and dimensions, specimen conditioning, casting details, and specimen geometry.

Fiber reinforcement is found to be very effective in reducing the permeability of unstressed concrete, a trend which occurs in conjunction with increasing fiber volumes. Figure 2-2 shows the relative permeability values for fiber-reinforced concrete and plain concrete without stress, as reported by Banthia and Bhargava (2007).

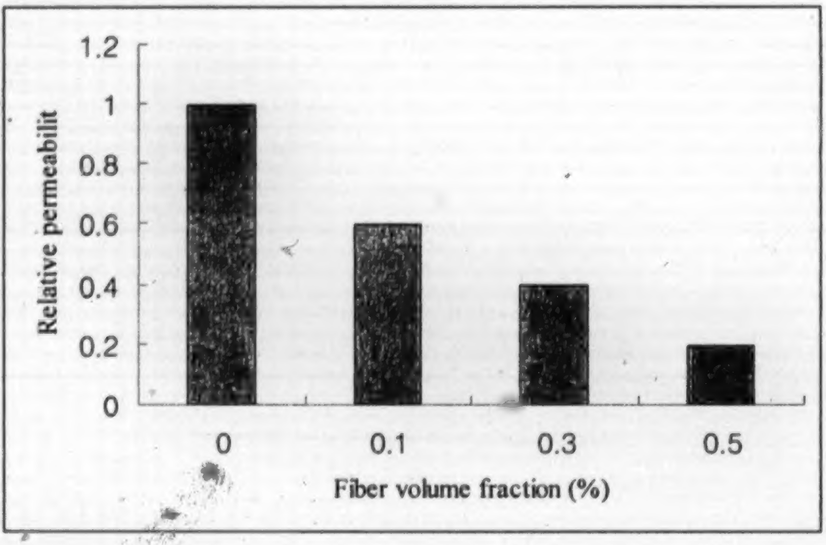


Figure 2-2 Relative permeability values for fiber-reinforced concrete and plain concrete without stress (Banthia and Bhargava, 2007)

2.3.2 Loading capacity of steel fiber reinforcement concrete

Altun et al (2007) carried out an experiment in order to summarize the mechanical properties of steel-fiber-added concrete (SFAC) and steel-fiber-added reinforced concrete (SFARC). They used C20 (Concrete strength 20MPa) and C30 (Concrete strength 30MPa) classes of concrete with the addition of steel fibers (SFs) at dosages of 0 kg/m^3, 30 kg/m^3, 60 kg/m^3, and measured their compressive strength, split tensile strength, moduli of elasticity, and flexural toughness.

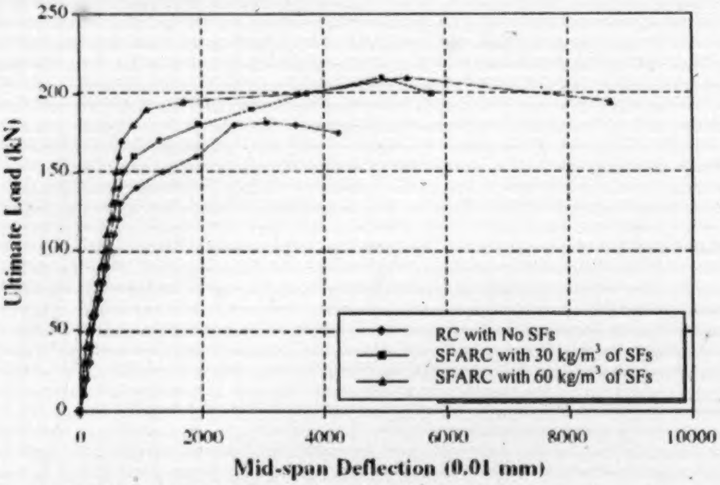


Figure 2-3 The average (ultimate load)–(mid-span deflection) relationships determined experimentally for the 3 groups SFARC beams with C20 class of concrete. (Altun et al., 2007)

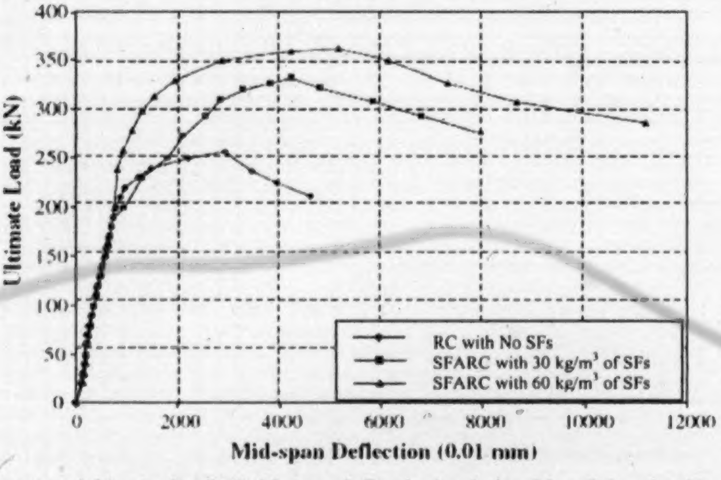


Figure 2-4 The average (ultimate load)–(mid-span deflection) relationships determined experimentally for the three groups SFARC beams with C30 class of concrete. (Altun et al., 2007)

Table 2-3 Results of the bending experiments on RC and SFARC beams (Altun et al, 2007)

Beam Sample	Concrete Class	SF dosage (kg/m^3)	Tensile steel (mm)	Theoretical ultimate load (kN)	Measured ultimate load (kN)	Experimental ultimate load)/(theoretical ultimate load)	Average of (experimental ultimate load)/(theoretical ultimate load) ratios	Toughness (kN mm)
C20-1,2,3-0	C20	0	2ø16	126.0	184.50~201.6	1.46~1.60	1.55	5495~5970
C20-4,5,6-30	C20	30	2ø16	126.0	201.90~210.0	1.60~1.67	1.63	27,550~29,501
C20-7,8,9-60	C20	60	2ø16	126.0	210.30~209.0	1.66~1.67	1.67	29,830~30,800
C30-1,2,3-0	C30	0	2ø16	148.6	250.90~262.30	1.69~1.77	1.74	9,925~10,965
C30-4,5,6-30	C30	30	2ø16	148.6	320.25~357.2	2.16~2.40	2.26	26,382~29,856
C30-7,8,9-60	C30	60	2ø16	148.6	352.95~370.45	2.38~2.49	2.45	29,460~30,045

As shown in Figures 2-3, 2-4, and table 2-3, the toughness of SFARC beams with 30 kg/m^3 of steel fibers increased 390% relative to that of RC beams (with no SFs), and yet the toughness of SFARC beams with 60 kg/m^3 of steel fibers was only 32% greater than that of SFARC beams with 30 kg/m^3 of steel fibers.

The increase in the actual ultimate load after the addition of steel fibers at a dosage of 30 kg/m^3 was 30% with respect to that of RC beams with no steel fibers, and the further increase was only 11% for a two-fold increase in the mass of steel fibers. These comparative findings seem that the SFARC with a steel fiber dosage of 30 kg/m^3 may be more effective and more beneficial than that with steel fiber dosage of 60 kg/m^3 in view of the flexural behavior of SFARC beams.

It is believed that SFARC beams having steel fiber at a dosage of about 30 kg/m^3 should be favored or even adopted in common practice, since the crack formation, crack size, and crack propagation in beams against bending moments are appreciably better. Furthermore, the ultimate bending-moment-carrying capacity is slightly better; and thirdly, the toughness is much higher than that of RC beams having the same conventional reinforcement but no steel fibers.

Bischoff et al. (2003) also tested loading capacity for slabs with equivalent amounts of either welded-wire reinforcement (WWR), fibrillated polypropylene fibers, or steel fibers using model slabs with fixed dimensions (2.5 m*2.5 m*150 mm thick) cast on grade in a test pit and loaded to failure. In this experiment, 0.4% (30kg/m^3) and 0.1% (10kg/m^3) steel fibers were used, along with 0.4% (3.6kg/m^3) and 0.1% (0.9kg/m^3) fibrillated polypropylene fibers, and single (0.16%) and double (0.45%) layers of WWR for comparison. Test results show that steel fibers are a suitable alternative to using properly positioned WWR.

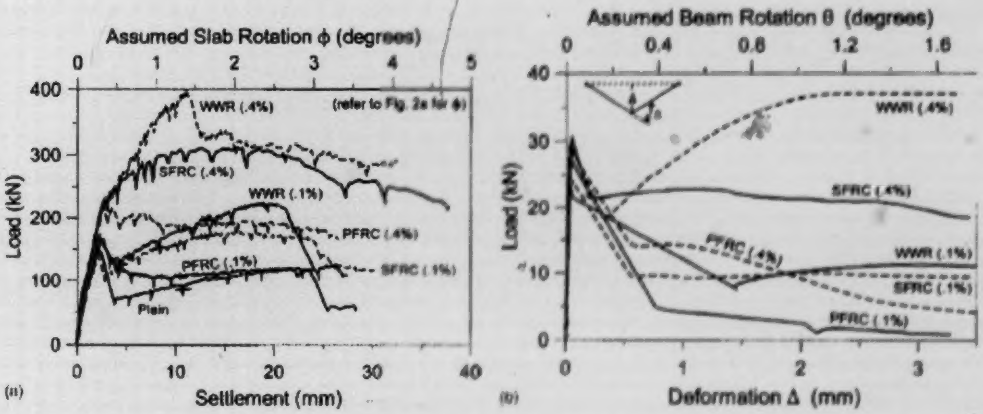


Figure 2-5 (a) Model slab test results (Bischoff et al, 2003). (b) Typical flexure beam test results (Bischoff et al, 2003)

As shown in Figure 2-5, test results indicate that SFRC can facilitate a load-carrying capacity comparable to that of properly positioned WWR, while fibrillated PFRC is *not* an effective replacement for WWR in ground supported slabs, especially when the reinforcement is intended for crack control of hardened concrete. In this respect, SFRC is expected to perform better than PFRC in the pressurized tunnel lining, while it is projected that both types of fiber reinforcements may be effective in reducing plastic shrinkage.

Mashimo.H et al. (2002) carried out two loading experiments. One was intended to generate the basic data needed to understand the mechanical characteristics of tunnel linings, which were made of plain concrete or concrete with steel fiber. The other was conducted in order to obtain actual data in a full-scale model test.

According to the first set of results, (1) the lining constructed with steel fiber tends to distribute cracks and has the effect of preventing the falling of concrete debris from the lining under each loading condition; (2) the lining with steel fiber does not stimulate an increase in structural strength under the condition that the axial force is dominant; and (3) the lining with steel fiber constrains the development of cracks and shows itself to be more stable than the lining made of plain concrete.

Through the full-scale model test, in the case where the influence of axial force was dominant, steel fiber had little effect on improving the load-carrying capacity. However, while the plain concrete lining showed falling concrete debris, no such phenomenon was observed in the case of the steel fiber reinforced concrete. They also showed that steel fiber reinforced concrete improved the load-carrying capacity under the condition that bending moment was dominant.

Kooiman A.G. et al (1999) investigated the applicability of steel fiber reinforced concrete in shield tunnel linings and showed that 60kg/m^3 high carbon steel fibers could replace the conventional reinforcement mesh. The production process of the prefabricated tunnel segments is divided into four stages. Workability in the mixing stage is decreased compared to a similar mixture without fibers, and mixing time is prolonged from 3 minutes for the conventional mixture to 5 ½ minutes for SFRC to ensure a homogeneous fiber distribution. Finishing the concrete surface proved to be more difficult for SFRC than for concrete without steel fibers, since the latter case involved practitioners contending with protruding steel fibers. This result may comply with the comments of ACI, 544.3R (2005). According to the evaluation of the installation, preventing cracking from high splitting stresses caused by thrust jacking forces is not even necessary. The research shows that the cracks will close as soon as the TBM pushes itself forward and the tunnel ring is compressed by combined loads from soil pressures, injection mortar pressures, and ground water pressures. In three of the twenty scenarios monitored, it was found that among conventionally-reinforced tunnel rings, excessive cracking and real damage was observed in elements next to the keystones, whereas in the SFRC tunnel section no damage appeared near the keystones. However, it is difficult to conclude just from this observation that steel fiber reinforcement segments perform better than the conventional tunnel assignments.

Roland de Waal (2000) carried out a pilot design of steel fiber reinforcement and demonstrated the possibility of reducing the thickness of the concrete lining in his PhD thesis. Although the lack of a suitable structural analysis model prevented his showing the exact lining reduction, a reduction of 0.05 m in the lining design (to a thickness 0.25 m) of the second Heinood tunnel can certainly be accomplished. And this reduction can save up to 2% of the total construction costs of the shield tunnel. However, it should not be assumed that such a reduction can be accomplished in every case, especially since the determination of the main reinforcement is still a weak point in the broader design. The current design is based on statically-determined beam loaded in bending, which does not take into account the post-cracking behavior of SFRC. A method for integrating the post-cracking behavior of SFRC and consequent redistribution of the stress is still under research. Nanakorn and Horii (1996) proposed a fracture mechanics-based design method for SFRC tunnel linings. Their rationale is that cracking and the resulting transmitted stress by fibers should be considered in the estimation of the maximum resultant forces of the critical cross section. Although fracture mechanics is based on experimentation, some assumptions must be introduced, and one of the main points to be clarified is the validity of the assumption regarding stress distribution. Specifically, suppositions are made about the relationship between the constant stress and the tensile strength carried by fibers along the crack. In reality, the transmitted stress diminishes with increasing crack opening displacement, according to the tension-softening curve.

The other point to be investigated is the assumption about maximum crack length. The estimated bending moment capacity increases along with crack length, and the tensile strength carried by the fibers is determined by a bending test. The central assumptions are that axial strain in compression is proportional to the distance from the neutral axis; that tensile strength carried by the fibers is considered in terms of the tensile stress in SFRC members; and that the maximum length of the crack is 70% of the thickness of the lining.

This design method has attempted to use fiber reinforcement as a structural component in place of a simple additive, such as aggregate. Such a trial will provide an excellent opportunity for the tunnel design team to reassess the process.

In light of all this research, it is apparent that fiber reinforcement can improve loading capacity, while preventing excessive cracking and permeability. These characteristics offer to ensure the long-term durability of the infrastructure, thus reducing the high public expenditure associated with the repair of infrastructures.

2.4 Tunneling in soft ground

Tunnels are constructed under a range of different geological and geotechnical conditions varying from hard rock to very soft ground. The TBM process is a step-by-step progression involving excavation, ground support, spoil mucking, and installation of the final liner. The use of a TBM is quite practical for boring hard rock, where the face of tunnel is basically self-standing. For soft ground, alternatively, the tunnel face is usually stabilized by pneumatic pressure, slurry, and excavated soil. Selection of a suitable TBM for soft ground should be careful and comprehensive, taking into consideration its reliability, safety, cost efficiency, and constructability. In particular, the geological condition along the tunnel alignment is primary factor to be considered in the selection of an appropriate machine. In soft ground, the geological and groundwater conditions affect the stability of the tunnel face. Figure 2-6 is the flow chart for selecting a TBM for soft ground.

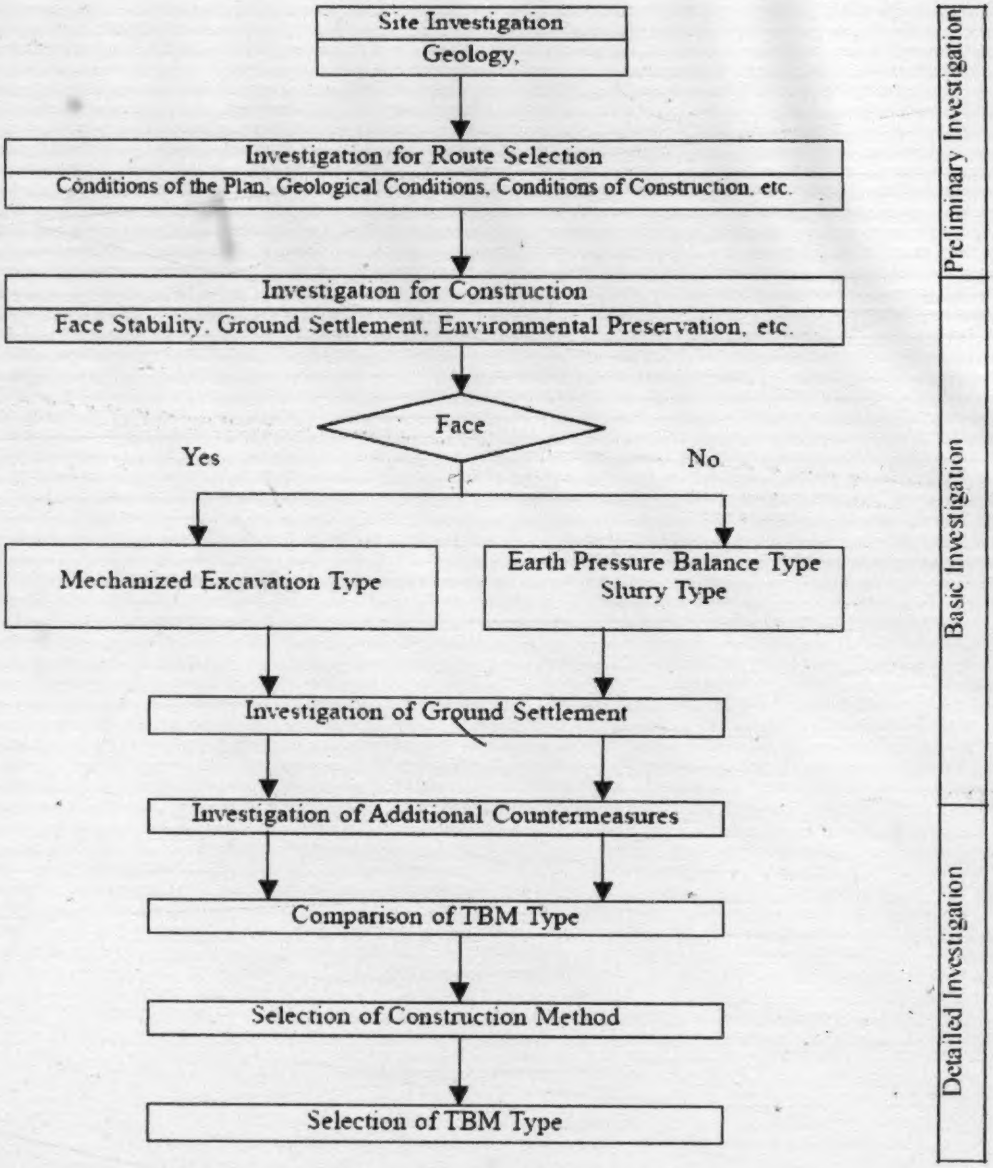

Site Investigation
Geology,
Investigation for Route Selection
Conditions of the Plan, Geological Conditions, Conditions of Construction, etc.
Investigation for Construction
Face Stability, Ground Settlement, Environmental Preservation, etc.
Face
Yes
No
Mechanized Excavation Type
Earth Pressure Balance Type
Slurry Type
Investigation of Ground Settlement
Investigation of Additional Countermeasures
Comparison of TBM Type
Selection of Construction Method
Selection of TBM Type
Preliminary Investigation
Basic Investigation
Detailed Investigation

Figure 2-6 Flow chart for selecting TBM for soft ground (Adapted ITA, 2000)

The closed-face machines for soft ground tunneling have become more sophisticated in recent years. As shown in Figure 2-7, there are generally three types of closed-face machines: Earth Pressure Balance Machines (EPBMs), Slurry Machines (SMs), and Compressed Air machine. The development of this machinery has resulted in tunnel projects with problematic ground conditions being tackled which had previously been too difficult to complete using more conventional methods. The EPBM is a shield machine which uses an earth pressure balanced face, and is stabilized at the working face through the creation of supporting pressure. In the case of the Slurry Machine, where mixed or unstable geological conditions threaten the stability of the tunnel face, the extraction chamber is filled with a pressurized liquid suspension material. The selection of machine for soft ground tunneling, either a slurry machine or an EPBM, is contingent on the given geological conditions. Compressed air machine had been the only face control technology available prior to the development of EPBM and SM techniques, but is now becoming obsolete because of the difficulties involved with working conditions and the so-called "caisson disease" associated with its application.

Furthermore, there is an increasing demand for soil conditioning related to the use of both EPBMs and SMs. Tunnel construction for sewers, for instance, often takes place in soft ground under urban areas and rivers.

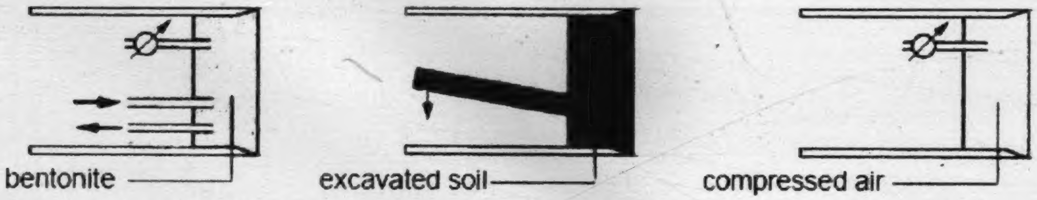


Figure 2-7 Face support by closed face tunneling a) Slurry (slurry shield) b) Excavated soil (EPB shield) c) Compressed air (Kovári and Ramoni, 2006)

2.4.1 Selection of a closed-face machine for tunneling in soft ground.

Both the EPBM and the SM were developed initially in Japan and Europe. In Japan, development of the SM began in the 1960s while EPBMs were introduced in the 1970s (The British tunneling society, 2005). These machines have undergone numerous advances and improvements since their first application. SMs were first developed for use in cohesion-less soils containing little or no silt or clay, whereas EPBMs were developed for application in weak cohesive soils. However, pure cohesion-less or weak cohesive soils are very rare, so application of the initial designs was both narrow and limited. Consequently it became essential to extend the application of SMs to cohesive soils and of EPBMs to cohesion-less soils. Having said that, the selection of which type of closed-face tunneling machine is to be used in soft ground is still a critical decision. The choice should be made only after a thorough assessment of the ground conditions and other factors anticipated.

2.4.2 Ground condition

Ground condition is a crucial factor in choosing a tunneling method. In many cases the ground conditions encountered along the tunnel route may vary significantly from the expected conditions. Fortunately, closed-face tunneling machines can be designed and manufactured to deal with a range of ground conditions. Some machines are able to cope a range of geological conditions with a minimal amount of reconfiguration and optimum operational efficiency. At this point it may be useful to note that there have been several attempts by tunnel experts to classify the range of naturally occurring soft ground characteristics. The British Tunneling Society, with reference to Whittaker and Frith (1990), summarized the range of ground conditions as follows:

1) *Firm Ground*: Tunnel construction can be advanced safely without initial support being required and the final lining can be installed before ground movement begins. Typical soil types are hard clay and cemented sand and gravel. A closed-face machine may not be needed in this ground.
2) *Raveling ground*: This type of ground is characterized by material that tends to deteriorate with time through a process of individual particles or blocks of ground falling from the excavation surface. Rapid raveling can occur below ground water and slow raveling can occur above it. Typical soils are glacial tills, sands, and gravels. A closed-face machine may be needed to provide immediate support to the ground.
3) *Running or flowing ground*: This ground is characterized by material such as sands, silts, and gravels in the presence of water, and highly sensitive clays that tend to flow into the tunnel as a viscous fluid. Above the water table this may occur in the form of granular materials such as dry sands and gravels. There will be considerable potential for rapid over-excavation in running and flowing ground. A closed-face machine will be needed to support ground safely unless some other method of stabilization is used.

4) *Squeezing ground*: The excavation-induced stress relief leads to ductile, plastic yield of ground into the tunnel heading in squeezing ground. This phenomenon generally is exhibited in soft clays and stiffer clays over a more extended period of time. A closed-face machine may be required to provide resistance to squeezing ground, even though in some conditions there is also a risk of the TBM shield becoming trapped.
5) *Swelling ground*: This type of soil tends to increase in volume as it absorbs water. This behavior is most likely to occur either in highly pre-consolidated clay with a plasticity index in excess of about 30, or in clays containing minerals naturally prone to significant swelling. A closed-face machine may be useful in providing resistance to swelling ground.
6) *Weak rock*: Weak rock may be taken into consideration effectively as a soft ground environment for tunneling because soft ground tunneling machines can be applied to such weak rock materials as chalk. As weak rock will often tend to be self-supporting over brief time intervals, a closed-face machine may not be required. On the other hand, the role of the water table may prove to be a significant issue. In some instances, use of a closed-face machine is an effective method of shielding the works against high volume water infiltrations that may be under high hydrostatic pressure.
7) *Hard rock*: Closed-face machines may also used in the context of self-supporting hard rock in order to guard against groundwater pressures and to prevent inundation of heading.
8) *Mixed ground conditions*: The most difficult challenge for closed-face machines may be to cope with encountering a mixture of different ground types either along the tunnel from zone to zone or within the same tunnel face. For longitudinal changes in ground conditions, a tunneling machine may convert from a closed-face pressurized mode to an open non-pressurized mode when working in harder ground types to avoid over stressing the machine's mechanical functions. Such an alteration may require some modifications to the machine. One common scenario, for example, involves a face with hard material in the bottom and running ground at the top. In this case, the machine will generally advance slowly while boring

the harder portion. Another problem may occur when a more competent layer exists over running ground in which possible over-excavation would create voids above the tunnel and below the competent material, giving rise to potential long-term instability problems.

2.4.3 Selection criteria between EPBM and SM

(1) Particle size and plasticity:

It is very important to determine the type of soil conditioning required before excavation begins, based on data obtained along the tunnel route. In general, sands and gravels are granular and are considered as non-cohesive soils, whereas silt and clay are fine grained and are classified as cohesive soils. The grain size distribution in soil is one of the decisive factors to consider when selecting the type of closed-face machine to be used. The favorable ranges of application for EPBMs and SMs are shown in Figures 2-8 and 2-9.

The SMs are ideal in loose water-bearing granular soils that are easily separated at the separation plant, but it has problems dealing with slits and clays. If the proportion of fines, (particles smaller than 60μm or able to pass through a 200 sieve), is greater than 20%, then the practicality of using an SM comes into question (The British tunneling society, 2005). In this case it will be the difficulty in separating excavated soil from the slurry, rather than the actual operation of the TBM, that is likely to drive up the contract and the operating cost.

An EPBM is a better choice where the ground is silty and has a high percentage of fines, both of which will assist in the formation of a plug for the screw conveyor and will control groundwater inflows. A proportion of fines below 10%, however, may be unfavorable for the application of an EPBM.

For SMs, the presence of higher Plasticity Index (PI) clays can lead to balling or clogging problems at the separation plant. Similarly, EPBM drives in clay can be an extremely difficult task if special soil conditioning measures are not taken. One potential problem is the clogging of cutting tools or the screw conveyor. Such high plastic clays may seriously slow excavating productivity or even bring the machine to a complete halt, and they also require a strong torque to turn the cutter-head and need more power consumption. In this case, clogging of the cutter-head occurs most frequently in the center of the head, rather than across the full face. Therefore, the selection of soil conditioning should take into account soil type while, in severe situations, anti-clogging agents should also be considered.

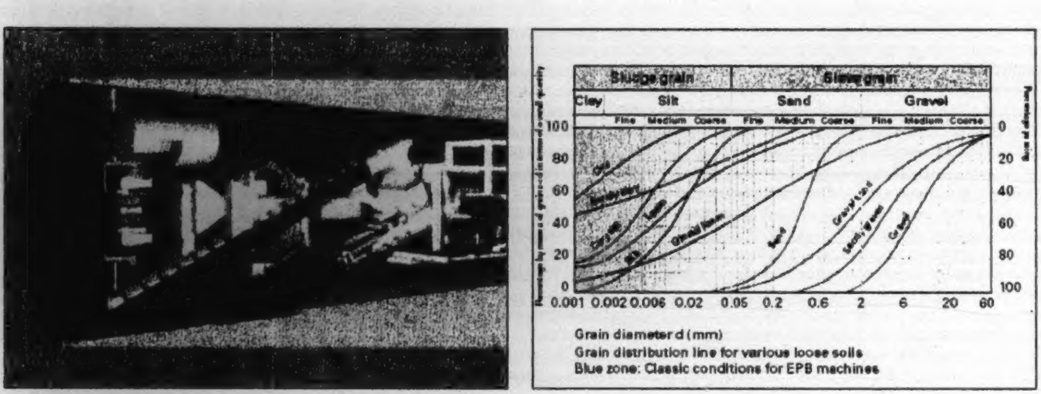


Figure 2-8 EPBM and ideal conditions (blue zone) (Munich Re group, 2004)

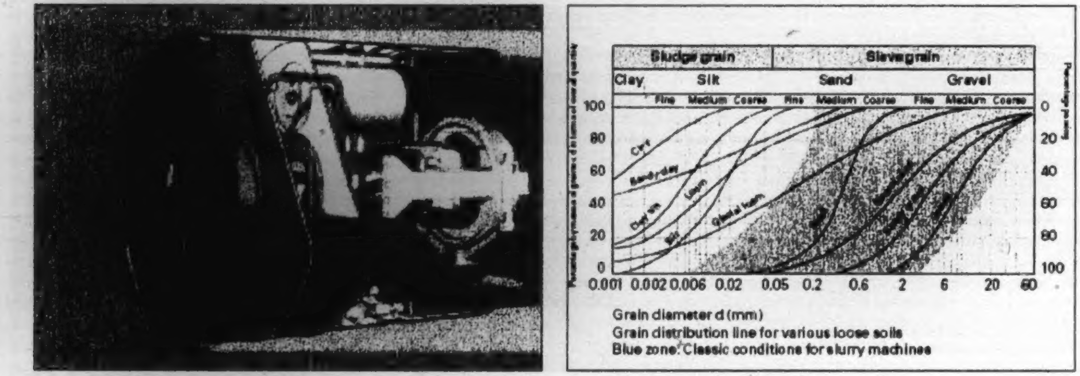


Figure 2-9 SM and ideal conditions (blue zone) (Munich Re group, 2004)

(2) Permeability:

The permeability of ground with respect to ground water is certainly a factor of practical importance. As shown in Figure 2-10, the dividing point in selecting between the two machines is a ground permeability of $1*10^{-7}$ m/sec. SMs apply to ground with a permeability exceeding this value while EPBMs are the better choice for ground of lower permeability. However, an EPBM *can* be used at a permeability greater than $1*10^{-7}$ m/sec if there is also an increased quantity of conditioning agent in the plenum. The selection should take into consideration the proportion of fines and the ground conductivity. Furthermore, according to the EFNARC the slurry machine can be applied to scenarios with a hydraulic conductivity (K) between 10^{-8} m/sec and 10^{-2} m/sec under varying charges of water.

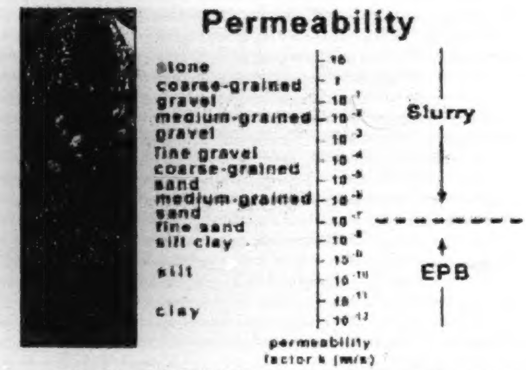


Figure 2-10 Applicable permeability for SM and EPBM <http://www.herrenknecht.de/index.php?id=505>

Although this permeability criterion is not absolute in every project, it can be the general standard for selecting machines. In general, tunneling projects can present an array of different types of geological conditions which are complicated and usually vary along the tunnel route. Indeed, the actual application is more dependent on the practical situation and past experience.

(3) Hydrostatic head:

Hydrostatic head with respect to tunnel alignment is an important factor affecting the stability of the excavation face and is of particular concern to the selection of a TBM and the successful conductivity reduction of the ground. In conditions where a high piezometric head is combined with high conductivity or fissures, it may be difficult to form an adequate plug in the screw conveyor of an EPBM. Under such circumstances, an SM may be the more suitable selection, as the bentonite slurry will aid to reduce stickiness of clogging soils in sealing the face.

(4) Settlement and excavated quantity:

Both types of closed-face machines can to some degree influence ground movements and stability as the machines advance. Both machines can be effective in controlling ground settlement during excavation. The quantity of excavation is an important control mechanism in the operation of both machines. This quantity enables practitioners to recognize over-excavation promptly and take action immediately to ensure ground control and stability. When using the SM, the quantity of spoil is measured by recording the density and flow of the slurry in the in-bound and out-bound pipe lines. For the EPBM, spoil is measured using weighers on the conveyor system.

(5) Face support:

Monitoring and control of the face support may be the most important issue in the application of either machine. The support medium of the face on SM is virtually a frictionless fluid (Z Einstein, 1989), comprised of a suspension of bentonite in water with appropriate additives. The slurry is prepared on the ground surface and circulated through a feeding pipe in order to support tunnel face. A significant feature of EPBM drives is that the earth or muck itself is used as the medium to exert support pressure on the face (Qiu Ling Feng, 2004).

The EPBM can successfully control and support a tunnel face in either a dry or a saturated fine grained soil, where no free water is present in the front chamber. On the other hand, the SM can reliably operate in essentially all types of soils – fine or coarse grained – with or without free water (Z Einstein, 1989). This feature is made possible because free water can be effectively countered by the pressurized bentonite slurry.

(6) Summary of evaluation criteria:

SMs should be applied mainly to non-cohesive soils with or without ground water present, whereas EPBMs are especially applicable to cohesive soils. For SMs, the proportion of ultra-fine grain (<0.02 mm) ideally should amount to no more than ≈10%, since higher quantities may lead to difficulties during separation. In the case of the EPBM, the proportion of ultra-fine fines (<0.06 mm) should amount to at least 20%, where the necessary consistency of the spoil can be improved by adding the appropriate conditioning agent (ITA, 2000).

2.4.4 Ground settlement caused by tunneling with EPBM

Any type of TBM induces some degree of ground movement as the machine advances through the ground. As seen in Figure 2-11, Kunito and Sugden (2001) show the general characteristics of ground settlement when an EPBM is used:

(1) Ground settlement (a) - Ground movements ahead of and above the face of the TBM are related to balance between the face and earth pressure. As such, the pressure in the mixing chamber must be kept within the required range.

(2) Ground settlement along the route of the TBM (b) - Ground settlements along the shield of the TBM are caused mainly by overcut in the vicinity of the cutting wheel. In order to decrease settlement, slurry can be injected into the gap between the shield and the ground.

(3) Ground settlement caused by the tail void (c) - Ground settlement is induced at the gap between the tail shield and the segmental lining. This tail void is often the primary contributor to ground movement with EPBM tunneling. The most effective method of limiting ground settlement is the proper grouting of the tail void.

(4) Ground settlement due to lining deflection (d) - This settlement is stimulated by lining deformation resulting from internal pressures plus the external ones caused from the ground and grouting.

(5) Ground settlement due to long-term movement (e) - The reason of long-term settlement is the eventual consolidation of the ground around the tunnel. Long-term settlement can be minimized through the prevention of ground water leakage.

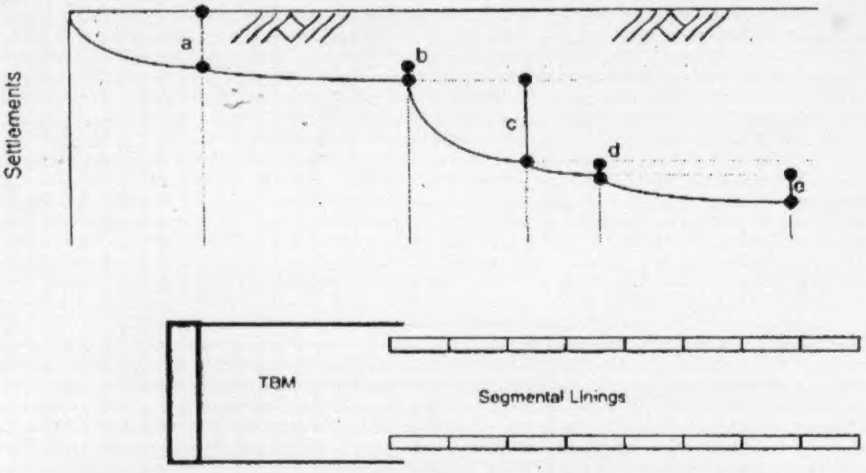


Figure 2-11 Ground settlement for EPBM (Kunito and Sugden, 2001)

2.5 Practical application of tunnel lining systems in Edmonton

2.5.1 South Edmonton sanitary sewer – SW1

SW1 is designed to convey sewage flows from developing neighborhoods in Southwest Edmonton to the Regional Wastewater Treatment Plant. The SW1 tunnel has an inside diameter of 2.3 m, with a length of 2.5 km, and with depths of cover ranging from 45 to 50 m, and was successfully completed in 2002.

In this project, a key geological challenge had to do with the existence of cemented sandstone stringers in the bedrock. In the recently completed sewer tunnel, TBMs encountered very thick cemented sandstone stringers with compressive strengths of up to 100 MPa. This required hand tunneling to break through the cemented sandstone stringers.

According to Bobey et al (2004), compressive tests for cores were carried out on 15 selected samples, and the results indicated that the compressive strength of weak clay shale ranged from 1 to 4.5 MPa while strengths of the weak sandstones and siltstones varied from 1.5 to 8.2 MPa. The sandstone ranged in thickness up to 300 mm and from 26.5 to 125 MPa in compressive strength. The tunnel's vertical alignment consisted mainly of weak clay shale bedrock. Here, the maximum compressive strength varied from 26.5 to 38 MPa in the selected tunnel zone. A shielded TBM equipped to erect a pre-cast segmental liner was selected to excavate the tunnel. The pre-cast liner was installed (see figure 2-12) and the tunnel completed without major incident.

The pre-cast segmental concrete lining was designed based on hoop stresses equivalent to a full vertical overburden load. Design for the tunnel lining was completed using a confinement convergence method, and on this basis a lower design load of 570 kPa was used, which was equivalent to about 70% of a full overburden load. During construction, load cells were installed between concrete segments to monitor load development on the liner. The load cells have been monitored for a period of about three years. The result is the determination that the actual lining load is equivalent to about 20% of full overburden pressure (Bobey et al, 2004). Compared to the design load, (i.e., 70% of full overburden), real acting loads on the tunnel lining are quite small. The monitoring of load cells indicated that the lining loads were slowly increasing at a proportion of approximately 3 to 8 kN/year over the two years (Bobey et al, 2004). Having said that, in order to achieve a more precise tunnel lining load, it should be monitored over a much longer period of long time. The resultant data will be integral to achieving a more economical tunnel lining design.

Figure 2-12 Pre-cast segmental concrete lining in SW1 project

2.5.2 South LRT in Edmonton

The South Light Rail Transit (SLRT) tunnel project from the University Station to the Health Sciences Station was finished in 2005 as a component of a multiphase project to extend LRT service to the south side of the city. The extension consists of twin tunnels with internal diameters of 5.8 m, each 290 m in length. About 40% of the Edmonton LRT system's 12.3 km consists of underground works successfully constructed using TBMs and a sequential excavation method. The SLRT is the most recent project to have passed under existing buildings and utilities and through a number of geological conditions. Although the portion of tunnel under discussion was relatively short in length, the difficult mixed-face ground conditions played a significant role in the decisions made to minimize the risk of ground movements under existing university's utilities and buildings. The geological conditions along the alignment varied from outwash sand and silt deposits, to glacial till with boulders, to soft bedrock under the water table. Under these conditions, two construction methods were available: the Sequential Excavation Method (SEM) and the use of a closed-face EPBM (Washuta et al, 2004). Finally EPBM was selected.

Construction of the portal began in March of 2003 and in the completed facility was operational by 2006. The tunnel lining of the SLRT system was composed of single pass, bolted, fully-gasketed, pre-cast segmental reinforced concrete rings (see figure 2-13). Each ring was made up of 6 pieces, including the key segment, and the segments were 250 mm thick.

One of the most challenging aspects of the project was the location, since the tunnel had to be completed with minimal ground movement under the university buildings and other facilities. As shown in figures 2-14 and 2-15, construction of the Edmonton SLRT extension was accomplished successfully using this lining system.

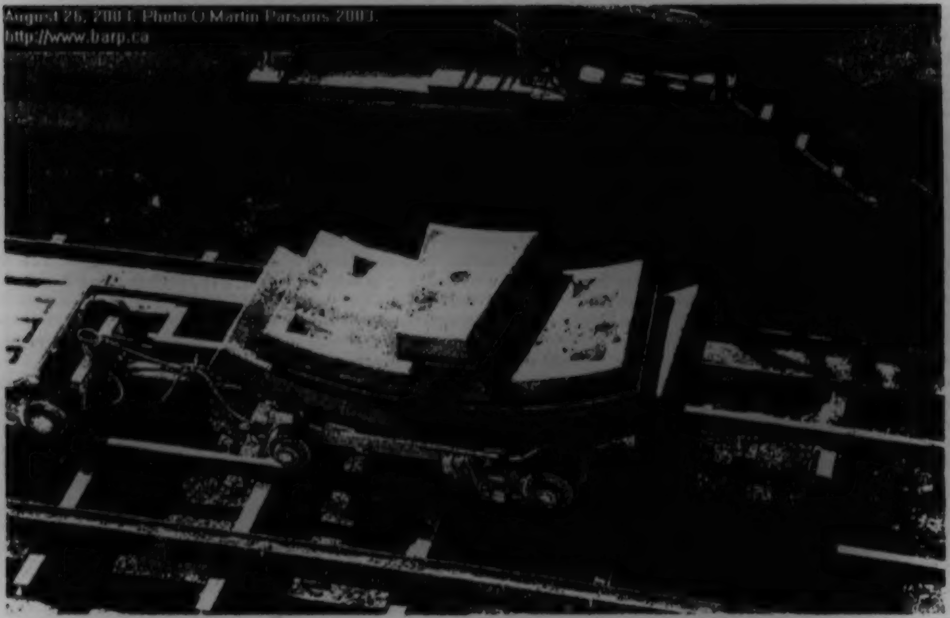

August 26, 2003. Photo © Martin Parsons 2003.
http://www.barp.ca

Figure 2-13 Bolted and fully-gasketed segmental concrete
<www.barp.ca/bus/special/etsslrt/TBM/tbmtrain3.jpg>

July 14, 2003. Photo © Dave Butler 2003.
http://www.barp.ca

Figure 2-14 EPBM and spoil removing <www.barp.ca/bus/special/etsslrt/TBM/071403-5p.jpg>

Figure 2-15 The completion of the tunnel

2.6 Performance indicators for project evaluation

In construction projects, performance indicators are used as tools for making informed decisions and quantifying project performance. The planning and design of a tunnel's lining should be approached using the optimum alternative, so selection of the best tunnel lining system is not carried out easily. A number of parameters may affect the design of the tunnel lining and will have diverse effects on the final product. The KPI working group (2000) recommends Key Performance Indicators (KPIs) for the benefit of the construction industry in the UK. The purpose of KPIs is to enable project planners to monitor project performance throughout the construction process (The KPI working group, 2000). KPIs can be categorized into seven main groups: time, cost, quality, client satisfaction, client changes, business performance, and health and safety.

More specifically, Matos et al has developed a set of performance indicators (PIs) for wastewater services, classified into the following six groups: environmental, personnel, physical, operational, quality of service, and economic and financial. These groups may also be divided into three priority levels depending on their relative significance in the evaluation process. The indicators in the first level provide a general overview of the efficiency of the wastewater services; indicators in the second level give deeper insights and detailed information; the third level consists of those indicators which provide the most specific and detailed information for evaluation. In this research, PIs use ratios such as % or $\$/m^3$ to represent all the relevant aspects of wastewater with respect to performance between variables, relying on the requirements of PI definitions. These PIs also highlight the possibility of generic applications in other field.

Soetanto et al (2006) also applied the performance criteria in order to evaluate the potential of hybrid concrete construction. The performance criteria in this research are divided into 7 dimensions, with each dimension consisting of several indicators. These criteria can also be classified as either Hard or Soft. The Hard criteria are quantifiable indicators, such as cost or speed of construction, and allow for a more objective evaluation, whereas the soft criteria are *qualitative* factors relating to individuals' experiences and perceptions. The qualitative criteria may be assessed as subjective factors and, as such, may lack the precision required for evaluation and consistent comparison in engineering decisions (Soetanto et al, 2006). As exhibited by the respondents involved in this study, this research demonstrates that experienced experts can provide high quality and valid information.

The performance indicators in this thesis are developed based on a literature review and refined based on the input of experienced experts. The experts involved in the W12 project have been recommended by the City's project manager and the selection was restricted to qualified experts from pertinent fields of study. Therefore, although the performance indicators are qualitative, the results of the evaluation are objective and serve to inform truly transparent decision-making, specifically for selecting a suitable tunnel lining system for the project.

Chapter 3

Tunnel lining systems for internal and external pressure

3.1 Introduction

A number of different temporary and permanent tunnel support systems are available for soft ground tunneling. The lining is often selected on the basis of operational criteria and is evaluated in the context of the construction environment prior to the project's being assessed in terms of specified ground loads (ASCE, 1984). Either of two tunneling machines were recommended for the construction of the W12 project, depending on the given geological conditions. An open-faced TBM was adaptable to the geological features of the northern portion of project, while an EPBM was required to support the tunnel face under the North Saskatchewan River. The City of Edmonton has also considered three types of lining systems: cast-in-place reinforced concrete, gasketed and bolted pre-cast segmental concrete, and pre-cast pipe. An evaluation of these three lining systems focuses on such issues as durability and water-tightness, the identification of an appropriate tunnel excavation method, the location of a shaft site to facilitate construction and operations for the permanent lining in order to minimize costs, and the task of assuring serviceability. In this chapter, the three types of tunnel lining systems are analyzed, taking into consideration design, construction, and issues related to internal and external pressure.

3.2 Description of the project

The W12 project consists of a 2.5 m (inside diameter) inverted syphon crossing the North Saskatchewan River from a north shaft location, near 85 Street and Jasper Avenue, to the existing McNally shaft at 84 Street and 106 Avenue, (see figure 3-1). The primary feature of the project area is the North Saskatchewan River valley, with a ground elevation ranging from 660-664 m in the uplands to 621-626 m in the river valley terrace. The North Saskatchewan River is about 130 m wide at the W12 tunnel crossing.

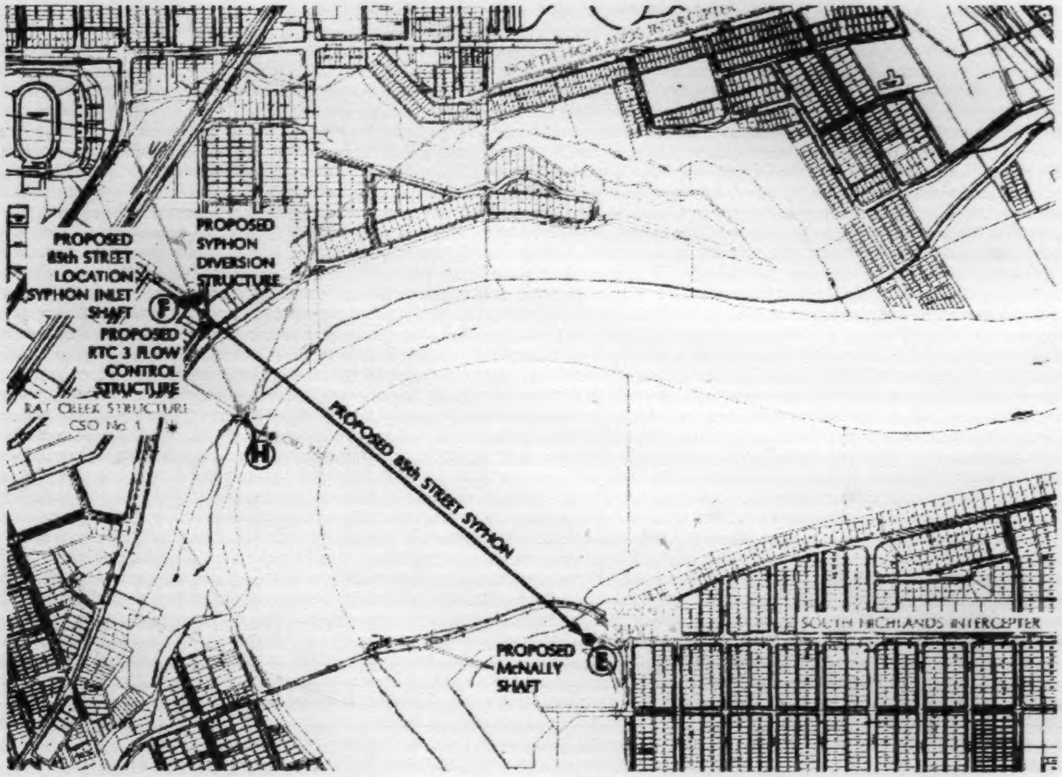

NORTH HIGHLANDS INTERCEPTER
PROPOSED 85th STREET LOCATION SYPHON INLET SHAFT
F
PROPOSED SYPHON DIVERSION STRUCTURE
PROPOSED RTC 3 FLOW CONTROL STRUCTURE
RAT CREEK STRUCTURE CSO No. 1
H
PROPOSED 85th STREET SYPHON
SOUTH HIGHLANDS INTERCEPTER
PROPOSED McNALLY SHAFT
E

Figure 3-1 Proposed inverted syphon tunnel (Preliminary design report of W12, 2005)

3.2.1 Geological conditions

As one can see from figure 3-2, this channel of ground under the North Saskatchewan River is incised into Upper Cretaceous bedrock, inter-bedded clay shale, sandstone and siltstone, with frequent coal layers and bentonitic clay seams. The clay shale would be described as very weak in terms of rock mechanics, with the siltstone and sandstone described as weak to very weak. The upper several meters of bedrock under the river channel are very weathered, as is reflected in the jointing and permeability of the upper bedrock. A number of coal seams ranging in thickness from about 0.2 to 1.6 m appear to be relatively continuous throughout the channel spanning the North and South shafts. Evidence was also observed of methane gas under pressure in the lower coal seams between elevations of 591 and 581 m. There is evidence of abandoned coal mine workings on both sides of the river, as reflected by voids, coal mine supports, and highly disturbed bedrock. The coal seam detected at an approximate elevation of 598-599 m would appear to represent the lowest limit of disturbance on the south bank of the river. On the north bank of the river there is also evidence of mine workings, but at a lower elevation of 589 m. In view of geotechnical aspects, the optimal vertical tunnel alignment to provide adequate bedrock cover over the tunnel crown is between 590 m and 598 m in elevation under the river channel. According to geological investigation, this tunnel alignment will encounter coal seams of varying thickness and may also encounter abandoned coal mine workings and disturbed bedrock on the north side of the river valley. Because it is also expected that practitioners will encounter coal layers with methane gas, proper ventilation and constant monitoring will be required during construction.

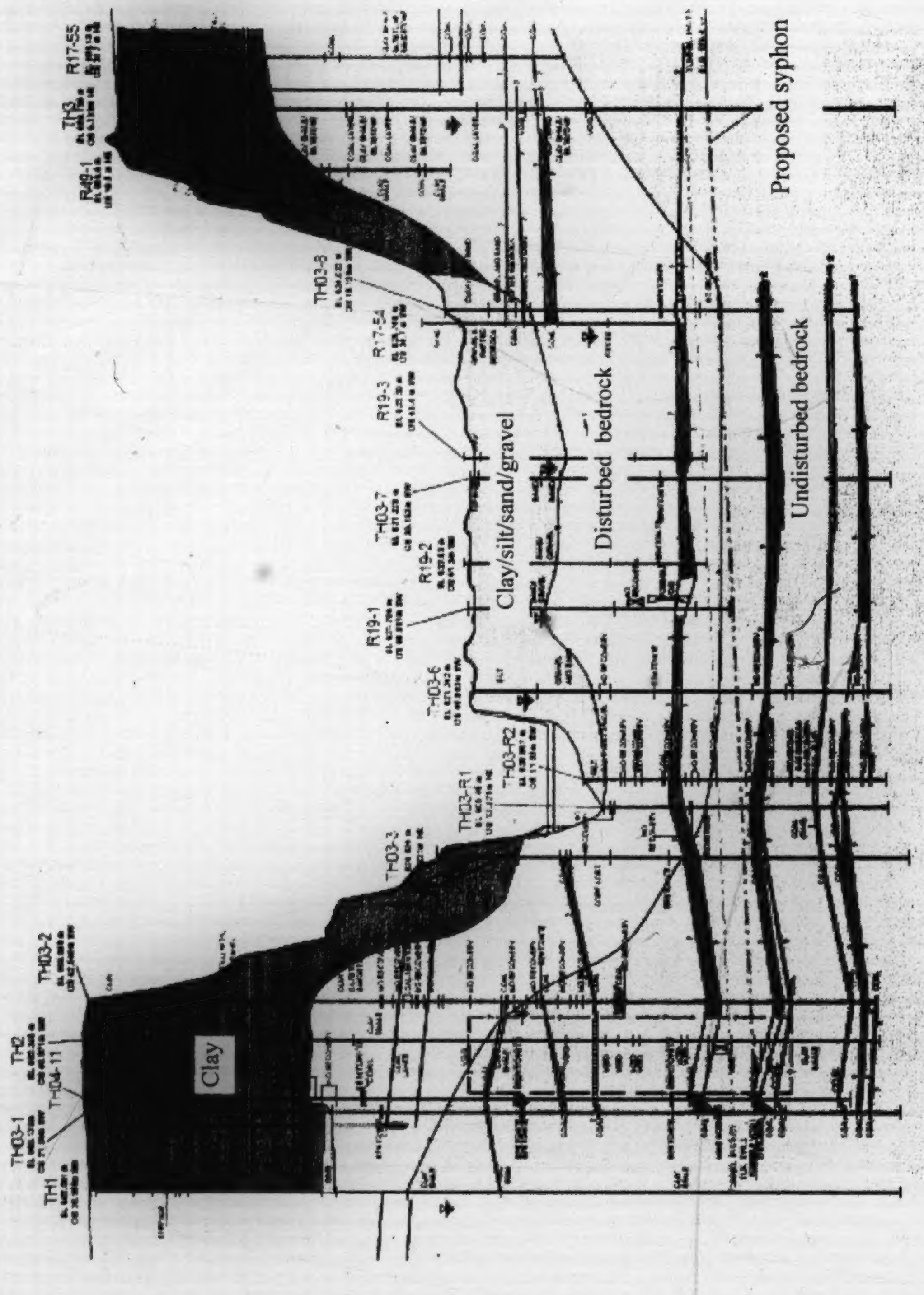

TH1
TH03-1
TH04-11
TH2
TH03-2
Clay
TH03-3
TH03-R1
TH03-R2
TH03-6
R19-1
R19-2
TH03-7
R19-3
R17-54
TH03-8
R49-1
TH3
R17-55
Clay/silt/sand/gravel
Disturbed bedrock
Undisturbed bedrock
Proposed syphon

Figure 3-2 Geological profile of W12 project (West Edmonton sanitary sewer stage W12-Preliminary design geotechnical investigation report, 2004)

3.2.2. Groundwater condition

The groundwater levels vary along the tunnel alignment from about 613 to 626 m. With tunnel crown elevations ranging from 593 to 598m, the groundwater level above the tunnel is expected to range from about 17 to 33m. The clay shale and bentonite sandstone are relatively permeable, and although seepage rates into the tunnel are expected to be low, greater seepage may be possible in the bedrock if water bearing coal seams or highly jointed bedrock are encountered during construction. Even as progress is being made on the open-faced TBM portion of the excavation, a significant amount of groundwater is appearing (and being pumped out) along certain sections of the channel. Even more groundwater is expected to be encountered once EPBM excavations begin and, accordingly, a number of countermeasures are needed.

The results of the permeability tests are displayed in table 3-1, ranging from $1.5*10^{-3}$ to $3*10^{-4}$cm/sec between elevations of 601.2 m and 599.7 m, and with a co-efficient in the range of $4*10^{-5}$ to $6*10^{-5}$cm/sec at elevations ranging from 598.1 m to 590.4 m. The higher permeability values likely reflect the presence of heavily jointed and weathered bedrock in the upper bedrock layer under the river channel.

Table 3-1 Summary of in-situ permeability and hydrofracture test results (West Edmonton sanitary sewer stage W12-Preliminary design geotechnical investigation report, 2004)

Test hole	Testing interval (m)	Estimated permeability (cm/sec)	Estimated Ko	Material
TH03 -R1	601.8-599.8	1 to $1.5*10^{-3}$	Too pervious for hydrofracture	Siltstone/Clay shale
TH03 -R1	593.8-591.8	Aborted due to no drop in head	1.2	Clay shale/siltstone
TH03 -R1	587.8-585.8	1.5 to $3.5*10^{-5}$	1.4	Siltstone/Sandstone
TH03 -R1	584.9	Aborted due togas in hole	Aborted due to gas & water in hole	Sandstone
TH03 -R2	594.1-592.1	1 to $2*10^{-5}$	1.4	Sandstone/Clay shale
TH03 -R2	588.1-586.1	Aborted due togas & water in hole	Aborted due to gas & water in hole	Sandstone
TH03 -R2	584.1-582.1	Unable to complete test	Unable to complete test	Clay shale/Siltstone/coal/Sandstone

The location of the water table in relation to the depth of the tunnel may prove to be critical to the performance of closed-face machines. An extreme and potentially adverse design situation in regards to leakage from the tunnel would exist where the applicable hydraulic grade-line is high above the ground level, and where the permeability of the rock mass is relatively high and the in situ groundwater level low relative to the invert level of the proposed tunnel.

The hydraulic conductivity of the ground is an essential design consideration for any tunnel. Groundwater inflows encountered during construction must be assessed, and are of particular import to the feasibility of cast-in-place concrete lining. Important data about permeability can also be obtained from previous tunnel excavations by observing groundwater occurrence in the tunnel.

3.2.3 Tunnel alignment

Two alternative tunnel plans have been considered for the project, stemming from the concept plan, risk assessment, and engineering workshops. One extends from an access shaft located at 85 Street and 106 Avenue on the north side of the river to the existing McNally shaft on the south side of the river, at a length of approximately 1225 m. The other extends from the permanent shaft located on the north side of the river valley near the existing Rat Creek Outfall to the existing McNally shaft, at a length of about 955 m (Preliminary design report, 2005).

The geological conditions within the tunnel's vertical alignment are largely analogous between the two options. The first alternative has been selected as, from a geological perspective, in this case both shafts are located beyond the river valley slope, whereas the second option requires the construction of a permanent shaft on the north valley slope. This alignment is not recommended due to existing slope stability concerns.

From a geotechnical vantage point, the vertical alignment of the selected tunnel is set below an elevation of 598 m in order to provide adequate bedrock cover over the tunnel crown. The vertical alignment is set so as to slope downward from the McNally shaft at an invert elevation of 595.4 m to the north shaft at 85th Street and 106th Avenue at an invert elevation of 590.3 m. Practitioners will likely encounter coal seams of varying thicknesses and, as mentioned, may even come across abandoned coal mine works and disturbed bedrock on the north side of the river valley. In summary, the tunnel runs primarily through weak clay shale and sandstone bedrock, but with a few interruptions of coal and methane gas.

As seen in Figure 3-3, the determination of a hydraulic grade-line may be an important design element of the pressure tunnel, and is expected to have an effect on long-term service-life. In fact, it is essential that long term serviceability of any water conveyance system be considered in the establishment of a suitable hydraulic grade-line. Two inlet sites in the W12 tunnel were proposed and reviewed. The first site, called the 85th Street Syphon, is located at 85th Street north of Jasper Avenue and the second site, called Rat Creek, is located immediately uphill from the Rat Creek CSO facility. The first inlet site was assessed in terms of the maximum capability of flow, and the expected operating velocities for the 2.5 m syphon at various operating flows are given in table 3-2 below (Preliminary design report of W12, 2005).

Table 3-2 Syphon (Diameter=2.5m) operating velocities (Preliminary design report of W12, 2005)

Velocity (m/sec)	Flow (m3/sec)	Grit particle rising velocity		
		Particle size (mm)	Rising velocity (m/sec)	Settling velocity (m/sec)
2.4	12	3	0.76	0.38
1.8	9	6	1.07	0.54
1.2	6	12	1.52	0.76
0.6	3	25	2.19	1.09

According to Table 3-2, the maximum particle size that would be able to be self-cleaned under maximum flows is 25 mm. Moreover, the maximum particle size that should be allowed into the syphon during any event is also 25 mm. Maximum flow velocity is less than 3 m/s and is thus acceptable for all types of lining systems.

Ensuring the smoothness of the liner surface, even at joints, may prove essential for the sewer tunnel. This velocity is given a Manning number n=0.013, a value generally applied to cast-in-place concrete. The velocity in the bolted and gasketed pre-cast segmental concrete linings, it should be noted, may be reduced due to the presence of bolt pockets and joints.

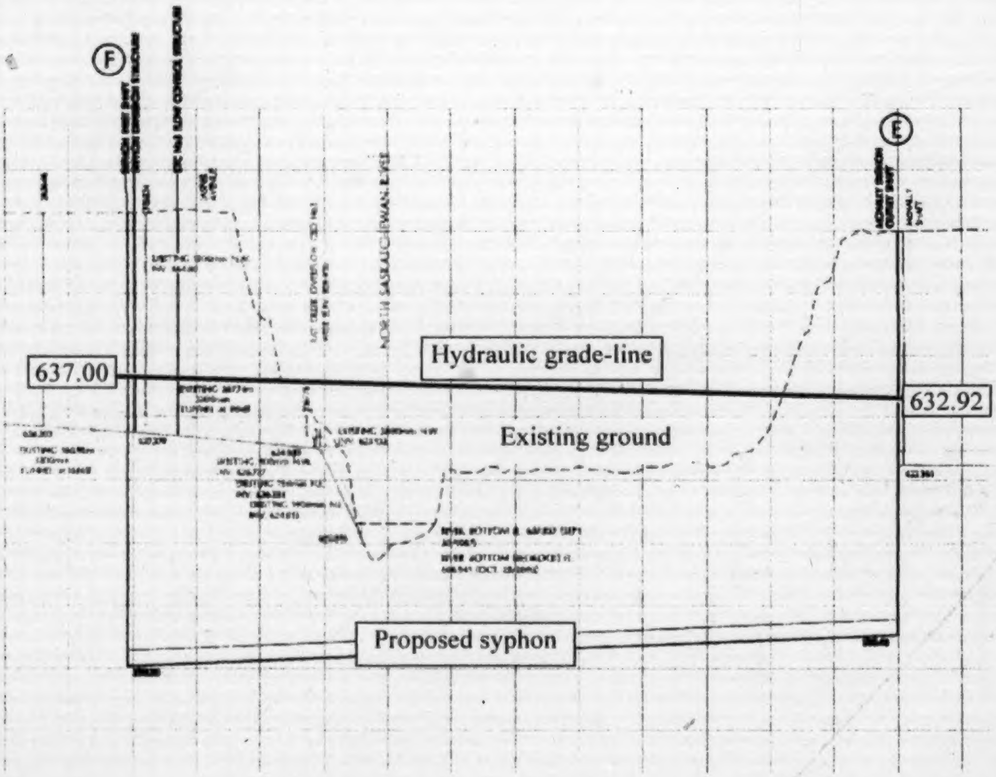


Figure 3-3 Hydraulic grade-line of proposed tunnel (Preliminary design report of the W12 project, 2005)

3.2.4 Overburden loads and internal pressure

A properly constructed lining behaves as a constrained arch, and stands to benefit from both the load sharing capacity of the adjacent ground and the lateral confining pressure on the arch. The tunnel lining in the north side of the W12 tunnel has been installed using a steel ribs and timber lagging system by the City's TBM. As seen in figure 3-4, the total pressures acting on the lining are varied depending on overburden pressure conditions. The secondary lining will be subjected to a range of operating conditions, from empty with no internal pressure to full with internal pressure, (the consequence of the water-head in the syphon). This tunnel lining system involves two governing cases with respect to construction and operation, depending on short- and long-term conditions. The City of Edmonton has reviewed the nature of overburden pressures, taking into consideration soil-lining interaction, operation conditions, and arching effect as follows (Memorandum, 2006):

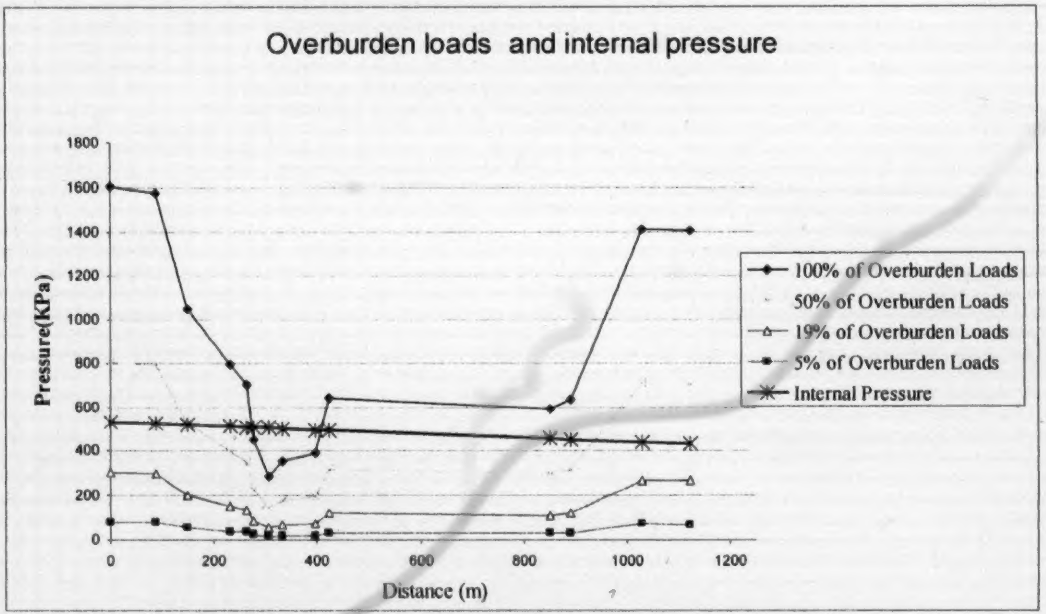


Figure 3-4 Overburden loads and internal pressure (Memorandum, 2006)

(1) Short-term condition: low earth pressures – high internal pressure

A relatively low external overburden pressure coupled with a high internal water pressure on the liner caused by the initial opening of the gate gives rise to a "low earth pressure – high internal pressure" situation. Under these conditions, the lining will have a net radial outward force causing tension in the liner. Based on measurements taken of the SW1 tunnel's segmental pre-cast concrete lining, the estimated equivalent uniform soil pressure acting on the lining is as follows (Memorandum, 2006):

- After 2 months, 100 kPa: equivalent to 12.5 % of full overburden pressure
- After 6 months, 120 kPa: equivalent to 15 % of full overburden pressure
- After 1 year 150 kPa: equivalent to 19 % of full overburden pressure

Short-term tunnel lining pressures on bedrock tunnels were as low as 8% of full overburden pressure, (the E.L. Smith tunnel at 66 m deep, for example), which is lower than the above noted values. Short-term soil pressures on the secondary lining at the W12 tunnel may also be relatively low, depending on the transfer of loads from the temporary steel ribs and lagging lining.

For design purposes, short term pressures operating within the first 6 months of tunnel construction should be assumed to range from about 5 to 8% of full overburden pressure. The lower end would be the most conservative in terms of calculating maximum tensile stresses in the lining. Horizontal design soil pressures acting on the liner may be based on a horizontal-vertical stress coefficient (k) of 1.

(2) Long-term condition: High earth pressure – zero internal pressure

We have this condition when the external soil pressure acting on the liner has increased to a maximal value, and the pipe is empty for servicing, resulting in a zero internal fluid pressure with maximum compressive stresses on the liner. In the long term, tunnel lining pressure will increase, but when the tunnel is empty, there will be no internal pressure and hence compressive stress in the liner will be at a maximum. Here, horizontal lining pressures may be based on a horizontal-vertical stress coefficient (k) of 1. The development of equal and long-term pressures on the liner relies on relatively uniform contact between the soil and the lining, as transferred through the temporary lining. As such, any voids behind the primary lining caused by over-cut or roof falls should be grouted up prior to the installation of the secondary lining.

The magnitude of the bedrock's stiffness may be of pertinence to the design of the pressure tunnel lining system. The stiffness of the rock affects any load sharing required of the reinforced concrete linings, which are designed primarily to control leakage from the tunnel. All lining systems must be evaluated in view of their effectiveness in supplementing the inherent strength and continuity of the ground, and construction methods must be deemed appropriate for sustaining the redistribution of stresses caused by excavation. A distinction should be made in that the *primary* lining is intended chiefly to establish a stable opening for construction operations, to protect construction personnel and control ground movement. In establishing the primary lining, we note that several geological and geotechnical characteristics of the ground are critical to the design and operation of the machine.

Tunnel lining systems generally do not support the direct total overburden soil or rock. In situ stresses are redistributed around the opening by virtue of the inherent shear strength and continuity of the ground. This effect is commonly referred to as arching effect. Theoretically, the lining has to support only those stresses not "arched" to the adjacent ground. But the influence of water pressure and discontinuities may have the result of reducing the shear strength. The prevailing state of minimum principal stress in case of rock mass is critical to the design of concrete lined (and non-lined) pressure tunnels, because of the long-term exposure of the rock mass to water pressures equal to the internal pressure. The linings of soil tunnels are acted on by stresses that result from soil-lining interaction. The vertical stress is generally directly proportional to the depth of the tunnel below ground surface and corresponds to the extent of the earth pressure.

3.3 Tunneling method

As described above, the excavation will primarily encounter weak clay shale and coal seams, with a wide range of strengths and geotechnical characteristics being observed along the alignment. One of the key construction phases will use an open-faced TBM, equipped to erect steel ribs and lagging for a temporary liner, from the site at 85th Street on the north side of the river right through to the temporary shaft. The EPBM will be necessary to handle with safety and efficiency the unstable geological conditions and high groundwater table anticipated along the portion from the temporary shaft on the north side to the existing McNally site. However, the nature of the permanent liner has not yet been decided for either zone.

The size of the excavation is decided based on a required conduit diameter and it may be subject very depending on whether or not a primary liner is required. In other words, for cast-in-place and pre-cast pipe lining systems, the tunnel diameter may be bigger than that for pre-cast segmental concrete since pre-cast segmental concrete lining does not need a primary lining. In addition, we note that the location of the shaft will affect the conveyance of fresh concrete and pre-cast segments. At the time of writing, a temporary shaft has been installed at the north side of the North Saskatchewan River. The length of this short tunnel is about 270 m, and the long tunnel is about 950 m from this shaft. Concrete may be modified for pumping for the cast-in-place liner and, obviously, requires more powerful pumping equipment for the longer distance.

The moderate upward slope of the W12 project may be desirable for the tunnel excavation and installation of lining, as gravity will direct groundwater inflows away from the excavation heading (i.e. the area of concrete placement). This condition also presents advantages in transporting the excavated soil, but carries disadvantages with respect to the prospect of using the self-weight of machine to facilitate excavation.

3.3.1 Tunneling with open-faced TBM

One feature of the open-faced tunneling machine is that it lacks the ability to seal openings at its front so as to prevent or slow the entrance of soil. Open-faced TBMs are generally used in competent soils showing reasonable stability. As such, areas where water-bearing, cohesion-less deposits exist present a problematic situation; moreover, the existence of groundwater complicates the construction procedure and necessitates countermeasures. In light of this fact, it is pertinent that the geotechnical analysis shows significant areas along the proposed tunnel alignments where groundwater is anticipated. It was thus concluded that, while an EPBM was appropriate for the preliminary stages of this project, an open-faced TBM should be employed following the installation of the temporary shaft. Although practitioners encountered a significant amount of groundwater in some portions of this section, the excavation was successfully completed. During construction involving the open-faced TBM, dewatering was carried out to counteract the pressures exerted by the groundwater and to prevent collapsing or flooding of the tunnel during excavation (see figures 3-5 and 3-6). Dewatering typically involves the installation of several wells along the affected portions of the tunnel alignment intended to reduce the groundwater level and, subsequently, the pressures expended by that groundwater. In the W12 tunnel, just one small pump was used for dewatering because the amount of groundwater was less than anticipated.

The open-faced TBM in soft ground tunneling can employ either of two types of lining systems: steel rib and lagging and pre-cast segmental concrete. In the case of the W12 project, Steel ribs and timber lagging was selected as the primary liner, erected within the tailskin of the TBM to stabilize the tunnel as the TBM excavates through the ground. After the tunnel drive has been completed, cast-in-place concrete is typically installed as a permanent liner. This process is known as a two-pass lining system. In our project, the tunnel drive and installation of rib and lagging in the northern portion have been completed, but permanent lining system has not yet been selected.

Open-faced TBMs have a minimal impact on traffic, facilities, and properties adjacent to the project. The only sign that construction is even taking place is at shaft openings where mining spoils are being removed and construction materials are being lowered into the shafts. These characteristics result initially in lower construction costs. In addition, use of the open-faced TBM accommodates the removal of boulders and other obstructions by manual methods during excavation. Over all, one can conclude that selection of the open-faced TBM for this section was both appropriate and economical.

Figure 3-5 Open-face TBM

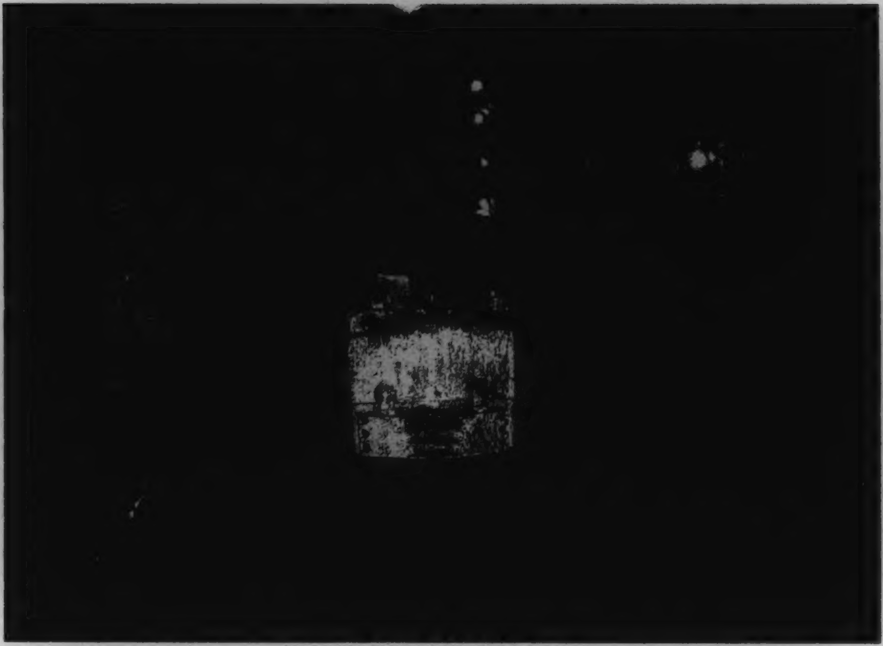


Figure 3-6 Mucker

3.3.2 Tunneling with EPBM

The EPBM is required to handle the unstable ground conditions and high groundwater table conditions anticipated in the southern phase of the W12 project. As seen in figure 3-7, the Lovat EPBM was selected to excavate the south side of W12. This EPBM was equipped with a mixed ground cutting-head enabled to interchange Lovat ripper teeth and disc cutters in order to deal with variable ground conditions. The EPBM operates by providing continuous pressure on the ground at the tunnel face, and by controlling the forward thrust of the machine and the rate at which soil enters and is removed by the screw conveyor. The consistency of the medium in the forward chamber is critical to maintaining the face pressure, and may be modified with the use of the Lovat ground conditioning system. This system adds water, foam, and other agents through ports in the cutting face, chamber, and screw conveyor in order to alter the characteristics of the soils.

Balancing the pressure in the tunnel face is essential in order to minimize ground movement and control surface heaves, (such as over pressure, for example). The segments will be installed using the erector but, again, the final lining system has not ye been decided.

Overall, the use of the EPBM for this portion of the W12 project is considered a feasible option under these particular geological conditions. Below is a brief outline of the technical specifications of the Lovat EPBM:

Model:	RMP 136RL/SE 22800
Type:	Multi mode EPB-Ribs/lagging, Expandable block
Cut diameter:	3.5 m
Shield length:	8.295 m
Overall TBM length:	53m
Cuttinghead:	Mixed Face/Flood doors up to 3 bar
Cuttinghead power:	450KW
Cuttinghead drive:	6 hydraulic motors
Cuttinghead rotation:	Bi-directional
Main bearing:	Triple axial roller
Max torque:	3.6 rpm @ 95 t.m
Min torque:	2.2 rpm @ 150 t.m
Peak torque:	190 t.m
Articulation:	2.5°
TBM propulsion:	12cylinder @ Max thrust 900 tonne 340 bar
Propulsion stroke:	1.676 m
Screw conveyor:	0.61 m diameter, 11 m long
Belt conveyor:	0.61 m wide, 35 m long
Segment erector:	Ring type
Gripping mechanism:	mechanical ball and cup
No airlock	
Total installed power:	736 KW

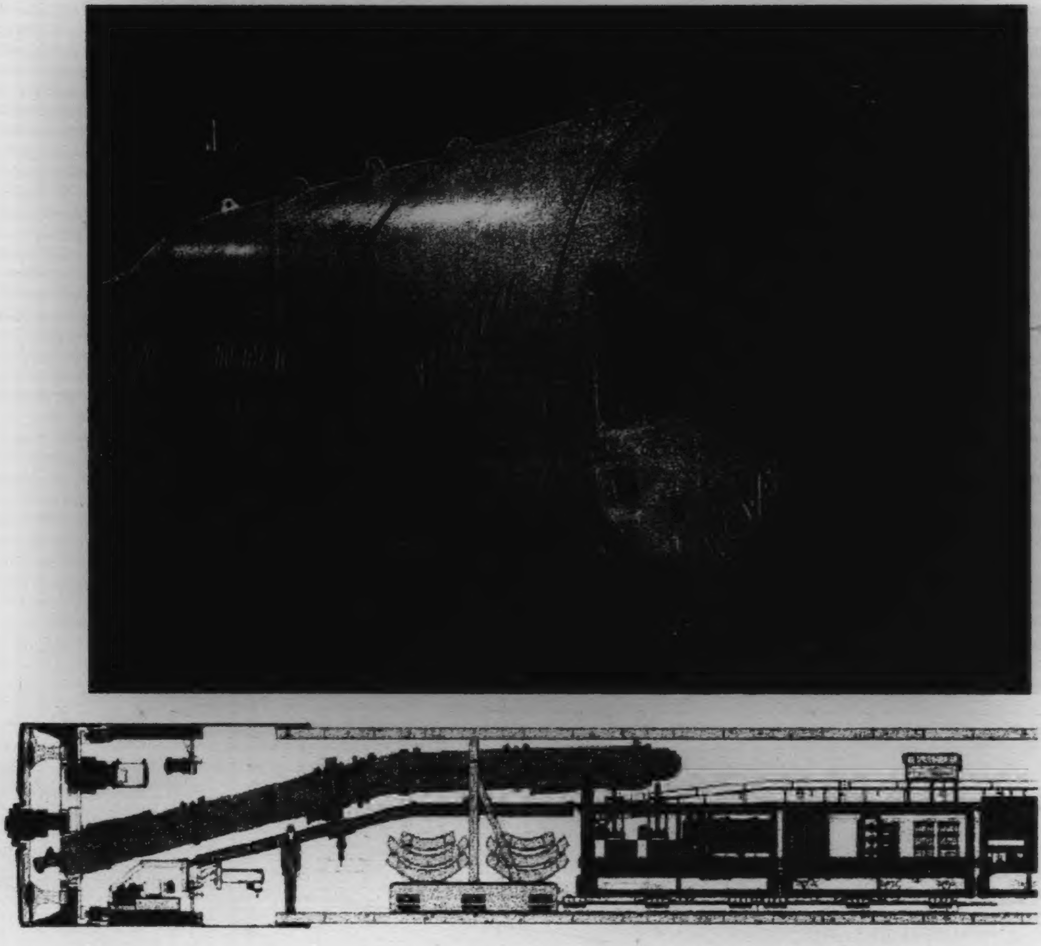


Figure 3-7 Earth Pressure Balance Machine (LOVAT)

The advancement of the machine through the ground is accomplished by use of hydraulic rams which thrust at the completed tunnel lining. The typical advance length of the EPBM is similar to that of the open-faced TBM: between 1 and 1.5 m, depending on the design and size of the segment.

3.4 Tunnel lining systems

3.4.1 Permanent lining requirements

An important dimension of the project in terms of the evaluation of tunnel lining systems is the hydraulic issues, such as water leakage and durability. Particularly, establishing a hydraulic grade-line will require surcharging the tunnel to an elevation of 637 m, which is approximately 25 m *above* the current river water level. Note that the internal water pressure exceeds the external pressure and that the tunnel is required to have a watertight final lining to prevent leakage. Three alternatives have been reviewed regarding internal and external pressure, and must be assessed with respect to risk, cost, and constructability.

3.4.2 Cast-in-place concrete with steel ribs and timber lagging

The City of Edmonton has used a cast-in-place concrete lining with rib and lagging until recently as it maintains its own bending machine and facilities for rib fabrication. As shown in figures 3-8 and 3-9, steel ribs in TBM tunnels are generally installed at the tail of the machine by hand or with the aid of a mechanical erector, and then a full circle of timber lagging is placed between the webs of each steel rib. Then, the TBM is advanced by jacking against the steel rib lining. The diameter of the steel rib set is slightly smaller than the excavated tunnel, but timber lagging does not allow for the grouting of voids outside the primary support until the final concrete lining has been placed. Moreover, ground settlement may occur and non-uniform loads may act on the ribs and lagging during the excavation phase, especially where soil features include large chunks or voids. Fortunately, almost all sewer tunnels have been installed under open spaces, rather than under tall buildings or important facilities.

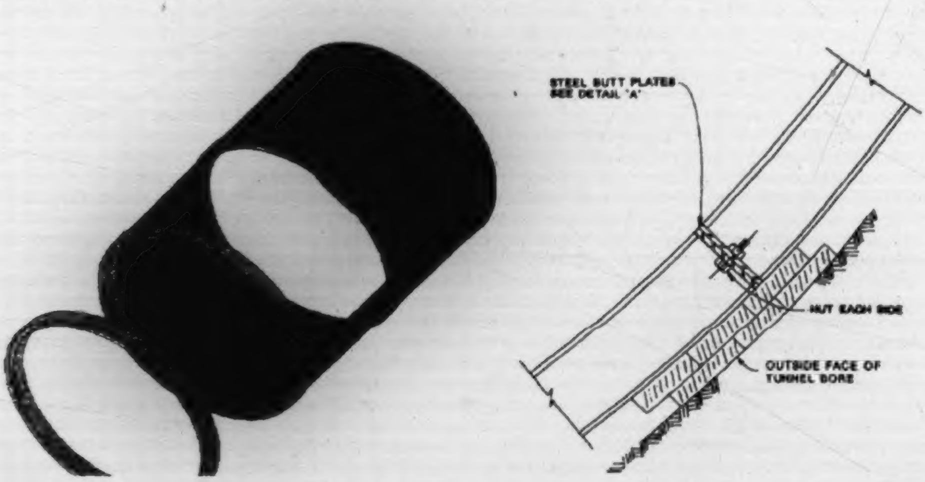


Figure 3-8 Typical steel ribs and timber lagging (American commercial incorporated) and rib joint (City of Edmonton)

Figure 3-9 Rib and timber lagging is being installed as TBM advances

Cast-in-place concrete lining provides a hydraulically-smooth inner surface, is relatively watertight, and is usually cost-competitive with steel rib and lagging as a primary lining. Concrete linings can generally be divided into two types depending on their function: non-reinforced concrete and reinforced concrete. The primary reason for using non-reinforced concrete lining is that it is fairly economic compared to other linings, and so is commonly used to deal with external water pressure, any applicable rock loads, and compressive requirements. In the case of the W12 project, (see figure 3-10), reinforcement bars should be installed to withstand internal and external pressures.

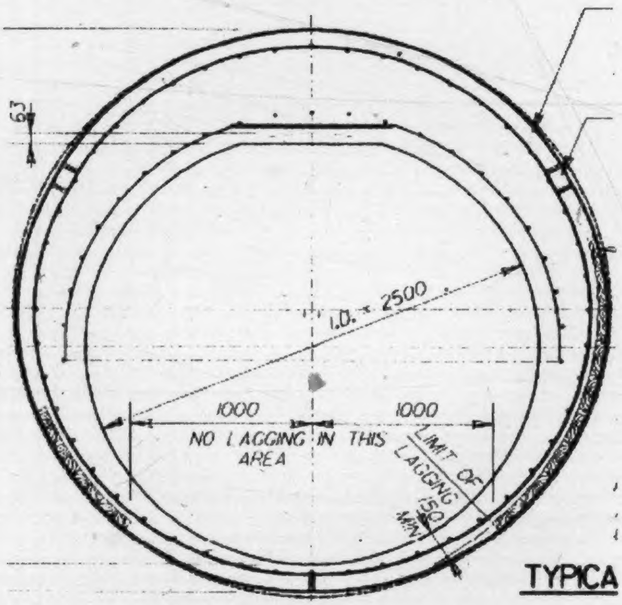


Figure 3-10 Typical cast in place reinforced concrete lining (City of Edmonton)

Concrete delivery and placement

The concrete is delivered to the installation site in the tunnel from the surface either by pumping or through a drop pipe. In many cases the concrete is pumped, and the pumping should continue to facilitate the delivery through the tunnel all the way to the point of placement. It is essential that the continuity and operating pressure of the pump does not segregate the concrete aggregates. Booster pumps may be used, and an appropriate pump type is selected depending on the distance. In the case of long tunnels, additional shafts or small holes may be needed along the tunnel to reduce the distance of concrete delivery. The pumped concrete should be selected with a relatively high slump to increase the pumpability and easy placement for long tunnels. The selection of slump and additives should consider the length of tunnel through which the concrete is to be transported and the behavior of the material under high pumping pressures. For long pumping distances, the addition of retarders or plasticizers can be helpful to maintain fluidity and reduce pump pressures.

The method of placement will determine whether the entire cross section is placed at one time, or is divided into two or more parts. The invert is usually placed first, depending on the tunnel size, and then the other parts. Discharging concrete from a placement pipe in the crown of the tunnel requires that the concrete flow down the sides of the form. A number of tunnels have been completed with the use of cast-in-place concrete with a crown placing port, but the segregation of concrete aggregate may occur if heavy reinforcement bar is installed. The constraints behind forms are related to ground support members and reinforcement bar and these constraints may also cause a segregation of aggregates. The maximum size of aggregate should be reduced for a lining which incorporates heavy reinforcement bars, as well as the diameter of the pumping line. One recent trend is that self-compaction concrete has replaced conventional concrete for cast-in-place concrete lining systems. Reinforcement bars are not necessarily needed in concrete linings, but in the case of pressurized tunnels, the need for reinforcement bars should be evaluated and the locations, (if any), specified in order to minimize restrictions on concrete placement.

Steel forms are the norm for tunnel construction, except where special shapes occur at turns and intersections (see figure 3-11). The forms are usually equipped with self vibrators, along with provisions in place to utilize internal vibrators through the inspection ports if necessary. The minimum time needed to obtain stability of the lining after concrete placement usually corresponds to the time necessary for the concrete to reach a minimum of 600 to 800 psi (4.2-5.5 MPa) compressive strength (ASCE, 1984). This strength can often be achieved within 8 to 10 hours after pouring, so form removal can usually begin after about 8 hours. One risk during the placement of the concrete, is that groundwater seepage into the tunnel may damage fresh concrete before it sets, and high-water flows may need to be pumped out before the concrete is placed. According to one concrete expert, the current state of groundwater flows does not pose a significant problem for the northern portion of the W12 project.

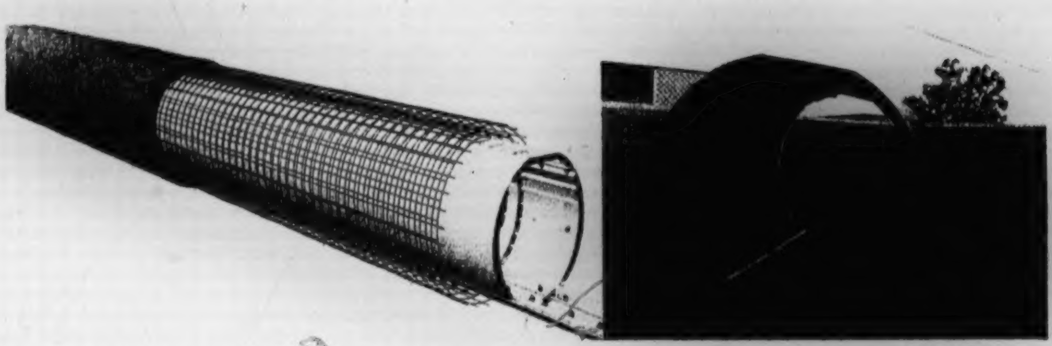


Figure 3-11 Model of reinforced lining and actual steel forms

Applicability

Cast-in-place concrete linings usually require support initially to secure the excavation. The design of a cast-in-place concrete lining is relatively straightforward if we assume the need for temporary support. Cast-in-place linings must be designed to support all the loads for the full design life of the tunnel. These linings are commonly designed for pressure tunnels, which require that a number of specifications be met in order to govern mix shrinkage characteristics, to protect fresh concrete from groundwater during placement, and so on.

In areas such that the bedrock has been found to be permeable, monitoring of groundwater infiltration is required during excavation. The tunnel lining will be bolstered with steel reinforcement to control leakage and the amount of reinforcement will be determined by the assessed modulus of deformation and applicable internal pressure.
In the case where a TBM has carried out excavation, cast-in-place reinforcement concrete lining is applicable since the tunnel length is short that installation of steel rebar and easy conveyance of concrete is quite plausible. The task is also eased by the fact that steel rib and lagging have already been installed. In addition, cast-in-place reinforcement concrete is a better system for preventing leakage of internal or external water than segmental concrete. One drawback of this option, however, is the reality of negative impacts related to labor concentration, small working space and requiring lots of time.

3.4.3 Pre-cast segmental concrete lining

Indeed, for the construction of a lengthy tunnel in rock or soft ground, a significant amount of time may be required to install lining using the more laborious cast-in-place method. The cast-in-place option is feasible, at least for a large-diameter tunnel and with enough time, but it is often not the most practical alternative. Conveying fresh concrete for a long distance can also be difficult, and can reduce the quality of concrete. In these cases, the use of a segmental concrete lining is a practical solution. The pre-cast segmental lining is installed by means of an erector as the tunneling machine advances, and the time of installation does not bear too significantly on the overall productivity of the process.

As we see in figure 3-12, a pre-cast segmental concrete lining is installed within the tail of a shield used to advance in soft ground. A key aspect of the design and construction of segmental lining is that it is assembled from several individual segments into a ring as the tunneling machine advances. Soft ground tunnels in Edmonton are most often constructed using shielded TBMs equipped with either pre-cast concrete segments or cast-in-place concrete with ribs and lagging. Under the groundwater table, the segments are generally bolted with gaskets to ensure water-tightness. Above the groundwater table, unbolted segmental linings are often recommended, but it is apparent that the W12 tunnel should be supported by bolted, gasketed segmental concrete lining due to the higher internal pressure.

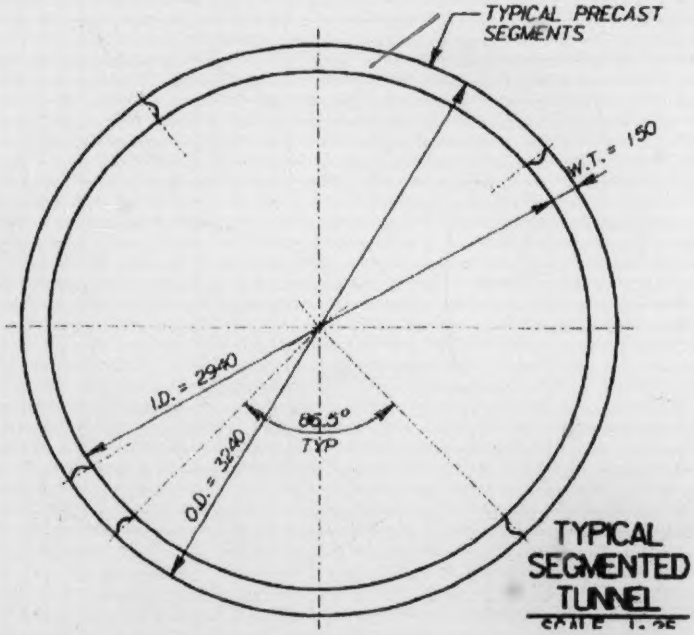


Figure 3-12 Typical pre-cast segmental concrete lining (City of Edmonton)

Design and construction considerations

The shield TBM or EPBM is usually advanced using jacks pushing on the erected segmental concrete lining. For this project, a segment ring will likely consist of six segments, and the ring divisions and dimensions must be optimized according to the project requirements. Segment rings are manufactured to be conical such that the ring end surfaces are not parallel, as inevitable deviations of the tunnel axis will occur during the excavation. The ring joints are stressed by thrust forces, while the transverse forces apparent in the ring joints result from the various deformations of neighboring rings. To avoid damage and to improve the load transfers, load transfer plates have been inserted in the ring joints and these have proven to be quite satisfactory. Geometrically, it has proven advantageous to bolt the segment connections along the longitudinal grooves and ring joints.

A gasketed and bolted concrete segment must be fabricated with great precision in order to ensure water-tightness, and this process inevitably extends the time required for construction. Before installation, the segments must be inspected thoroughly for damages and to ensure proper installation of the seals. Once a tunnel lining system has been selected, the compatibility between tunneling methods and equipment must be considered so as to match the specific needs of each system. The erector must match the pick-up holes in the segments and be able to rotate the segment into its proper place. The erector must also be equipped with any of the directional motion capabilities required to place the segment within the specified tolerances.

Water-tightness of segmental concrete lining

It is true that the selection of a segmental lining system is based on considerations of cost, constructability, compatibility with excavation machine, and other details involved in the construction process. However, these factors are more related to the construction stage than to the maintenance stage. With respect to the maintenance of the tunnel lining, functional and operational criteria must be satisfied prior to construction, especially water-tightness in the presence of higher internal pressure. In fact, a water-tight lining is difficult to achieve using segmental concrete without gaskets and bolts. In some cases, sealing strips or caulking are employed to retain grout filling, but the lining will not be equipped to endure high internal or groundwater pressures. For the W12 tunnel lining system, fully gasketed and bolted segmental concrete should be used to prevent water leakages. However, the success of this alternative depends on the acceptability or specification of allowable water leakages from the tunnel lining during operation, and also on the feasibility and cost of grouting during construction.

Water flow and velocity criteria within the tunnel often require a smooth lining surface, in which case segmental lining presents certain disadvantages compared to cast-in-place concrete.

Pre-cast segmental concrete must also be designed to resist transport and construction loads. During storage and transport, segmental concrete is typically stacked with strips of timber as separation. Lining rings used as a reaction for shield thrust force must withstand the distributed loads from the jacks, including eccentricities resulting from mismatching neighboring rings. Joint details must be reinforced to resist the chipping and spalling caused by erecting impact, stack, and uneven jacking on inaccurately placed segments. Tongue and groove joints are particularly vulnerable to spalling, and the edges of the grooves may require reinforcement. Interestingly, chipping and spalling in the joints may be the primary cause of leakage. Durability of the completed structure requires consideration of long-term corrosion and abrasion effects so, for a one-pass segmental lining, a high-strength concrete is usually desirable because of its strength, density, tightness, and durability. Steel reinforcement should also be employed to support internal and external pressures.

For the W12 project, bolts and gaskets will be necessary to prevent water leakage. However, bolted connections are not essential to the *stability* of the tunnel ring. In some instances, bolting *may* be essential, and may be deemed particularly desirable below the groundwater table or in cases of high internal pressure in order to prevent groundwater infiltration or exfiltration. But in most cases, under favorable conditions, i.e. SW1, unbolted linings are fully acceptable and no bolting is needed to ensure stability. In fact, in many tunnels the use of bolts is cause for concern. However, bolting between segments should be performed in order to guarantee *water-tightness*. Unfortunately, the process of installing and tightening bolts between segments is particularly labor-intensive and expensive, and adding the gaskets is also bothersome as the forms must be modified to accommodate them. Overall, the inclusion of bolts and gaskets make pre-cast segmental lining more costly, but both are necessary to prevent water leakage.

Damage of segments

It is plausible to observe numerous cases of partial damage on each segment occurring during transport from casting in the mould to placement in the tunnel. When using segmental lining, it is necessary to detect external or partial damage as soon as possible. After removal from the mould, a preliminary optical examination of the segment is necessary in order to determine if a given segment is good, repairable, or eliminable. Damaged Segments in any stage from production to erection must not be used for assembly in their damaged status. Slightly or partially damaged segments have to be renovated and repaired to ensure the durability of the structure, and heavily damaged segments must be removed. Georg and Davorin (2004) classified common damages and repair measures as follows:

(1) Cracks: Micro-cracks (generally smaller than 0.2 mm) within the groove need no repair since being filled by glue and damage cracks within the groove may be penetrated with epoxy resin of low viscosity.

(2) Spalling: Repairs for spallings within the edges of the groove are conditioned by depth and defined respectively in terms of maximum length. If spalling exceeds 5 mm in depth 20 mm in length, these segment edges should be repaired prior to use. Instances of spalling greater than 3 cm require repair with an epoxy resin to reconstruct the original geometry.

(3) Breakage: Breakages can be distinguished as occurring either within the groove or within the contact area. One repair measure is to stop with cement bound mortar. Breakages smaller than 5 mm need not be repaired under special requirements.

(4) Pockets: Locating pockets necessitates a careful checking of the concrete segment. Pockets can be treated and repaired just like breakages when they occur outside of the groove basis and do not reach the reinforcement bars. Pockets can be repaired by cleaning the structure reaching the intact concrete zone and filling with cement bound mortar.

(5) Joints: It is *not* necessary to dismantle the structure into segments in this case, and repair measures involve widening and filling the joints.

The reduction of damage during construction plays an essential role in increasing reliability and long-term performance.

Applicability

Pre-cast segmental concrete lining systems are perhaps the most common type of lining for soft ground tunnels, particularly for relatively long distances where the economics of using a TBM are most advantageous. The design of the segmental ring not only requires a structural analysis for the ground loads and the TBM thrust loads applied to the segments, it also requires the designer to consider the full process of manufacturing, storage, transport, handling, and erection, as well as the stresses generated through the processes of bolting and sealing. The thickness of linings for tunnels must satisfy design criteria to ensure water tightness for internal external pressure as well as safe handling during construction.

A bolted and fully-gasketed pre-cast segmental reinforced concrete lining in pressurized tunnels is generally used in conjunction with shielded TBMs, as mentioned, and this type of lining system may also be employed for the EPBM portion of the W12 project. However, the pre-cast segments require specialized manufacturing techniques to ensure quality and precision and accurate installation. Too many joints and bolt pockets can be detrimental to the smoothness and durability of the lining, and this fact should also be considered during evaluation.

3.4.4 Pre-cast concrete pipe lining

Pre-cast pipe lining is less complex than other methods, but it requires greater accuracy in its manufacturing, minimal to no damage from transporting and handling, and the elimination of gaps at the joints. Construction methods for installing pre-cast concrete pipe in tunnels depend on the size of the pipe, the length of the tunnel, and the type of primary lining used. For short tunnels, such as those under a highway or railroad, it is common practice to slide the pre-cast concrete pipe through the primary lining. For the W12 project, transportation of the pre-cast pipe through the tunnel will be facilitated by the muck rail and special equipment designed not only to transport the pipe but to fix it into place. Testing should be carried out for leakages at the joints as the pipes are installed at regular intervals according to the specifications. Assurance of water-tightness at the joints is of utmost importance for this lining system as joints may occur as frequently as every 1.2 m if this system is selected. Typically, the joints in this system include the spigot with O-ring gaskets installed between them, accessible from the interior of the pipe. After matching the joint, the installer connects an air line from a small air tank to the test fitting and pressurizes the space between the gaskets. Any leakage can be detected instantly, although the pressure is usually maintained for approximately five minutes. The annular space between the pre-cast pipe and the primary liner should be filled with grouting, either through ports installed in the pipe walls or by use of a grout placement line extended back to the remote bulkhead. The grouting placement line is pulled out as the grouting progresses.

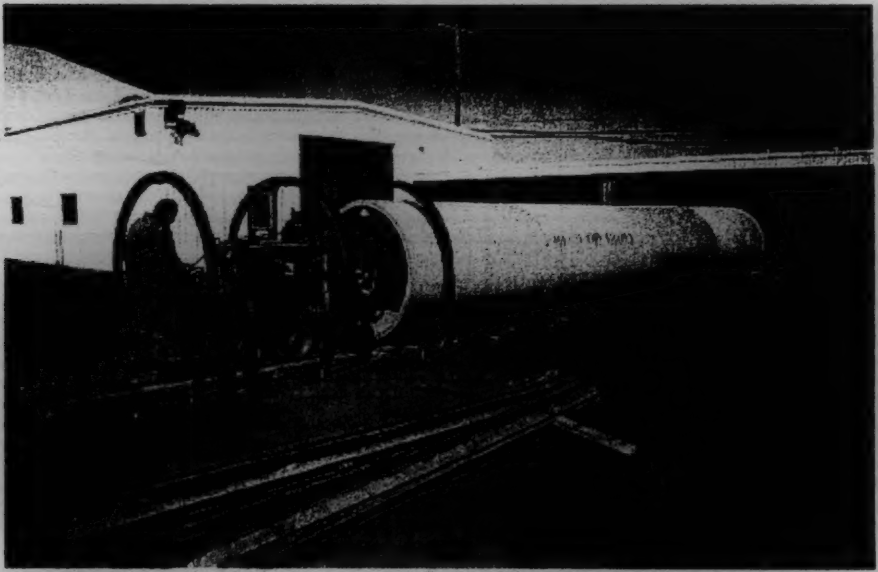


Figure 3-13 Pre-cast concrete pipe installation (City of Edmonton)

As with the Rossdale Water Intake tunnel constructed by the City, pre-cast concrete pipes will be installed using a pipe carrier or other equipment (shown in figure 3-13) after the tunnel has been completely excavated and the primary lining installed. For the north portion of the project, the tunnel is supported by steel ribs and lagging with an outer diameter of about 3.0 m. The gap between the pipe and ribs extends to about 25 cm and should be filled with concrete or some other material. This narrow space may serve to reduce productivity during the carrying and installing of such large pipe.

Pipe joints

Pipe joints may be one of the most important determinants affecting water-tightness, and they carry out a variety of functions depending upon the type of pipe used as well as its application. Prior to the selection of the proper type of joint, requirements for performance should be considered. Generally, joints are designed to provide the following (ACI 224. 3R, 2005):

(1) Resistance to infiltration of ground water and/or backfill material

(2) Resistance to exfiltration of sewage or storm water.

(3) Control of leakage from internal or external heads.

(4) Flexibility to accommodate lateral deflection or longitudinal movement without creating leakage problems.

(5) Resistance to shear stresses between adjacent pipe sections without creating leakage problems.

(6) Hydraulic continuity and a smooth flow line.

(7) Controlled infiltration of ground water for subsurface drainage.

(8) Ease of installation.

The real field performance of pipe joints depends primarily on the inherent performance characteristics of the joint itself, the severity of the operation conditions, and the employment of proper installation procedures. A number of different joints are used on pipe construction sites depending on performance requirements. Prior to the selection of a particular joint, it is usually necessary to compare the installation costs of several types of joints, as they may vary in both cost and in inherent performance characteristics.

The concrete pipe design manual goes on to summarize a number of pipe joints as follows (American concrete pipe association):

(1) Joints with mortar or mastic packing (Figure 3-14): In cases where leakage is an important consideration, these joints are not generally recommended as they do not inherently ensure water-tightness, but depend exclusively upon the workmanship of the contractor. Joints employing mortar joint fillers are rigid, and any deflection or movement after installation will cause cracks and permit leakage.

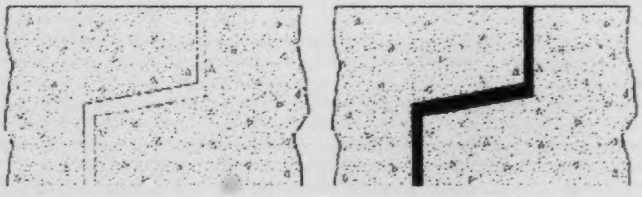
(2) Joints with compression-type rubber gaskets (Figure 3-15): A compression-type gasket may be used to seal concrete surfaces, with or without shoulders on the tongue or the groove. There is wide variation in joint dimensions and gasket cross section for this type joint but most may be used with either bell and spigot or tongue and groove pipe.

(3) Joints with O-ring gasket (Figure 3-16 and 3-17): These joints are basically designed for low pressure capability and are frequently used for irrigation lines, waterlines, sewer force mains, and gravity or low head sewer lines where infiltration or exfiltration is a factor in the design. They can prevent water leakage in both the straight and deflected positions.

(4) Steel end ring joint with spigot groove (Figure 3-18): This technology is

commonly recommended for a high pressure joint designed for use in water transmission and distribution lines, such as irrigation lines or sewer force mains. This joint ensures great shear strength and excellent water-tightness and flexibility, but can sustain being subjected to only a very limited amount of damage during transport and installation.

It is virtually impossible to define precisely the field performance characteristics of each of the joint types, but consultation with local manufacturers and constructors will provide information on the availability and cost of the various joints. Based on concrete pipe manufacturers' information and on an evaluation of groundwater conditions, the specifications should define allowable infiltration or exfiltration rates and/or acceptable joint types. The City of Edmonton has generally used a method which involves the spigot groove type joint with O-ring gasket.



Mortar packing Mastic packing

Figure 3-14 Typical cross sections of joints with mortar or mastic packing

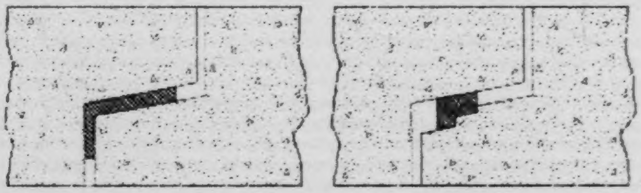


Figure 3-15 Typical cross sections of basic compression type rubber gasket joints

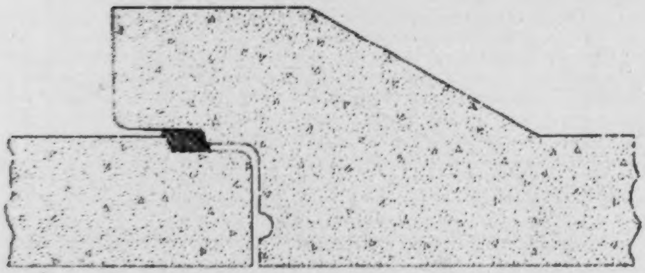


Figure 3-16 Typical cross sections of opposing shoulder type joint with O-ring gasket

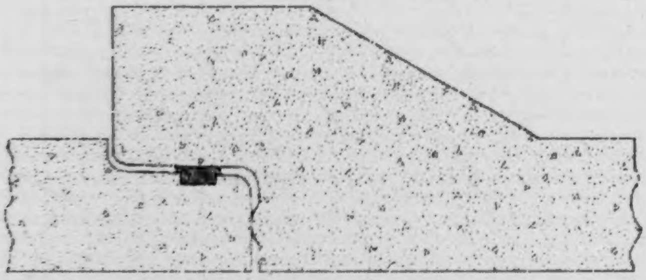


Figure 3-17 Typical cross section of spigot groove type joint with O-ring gasket

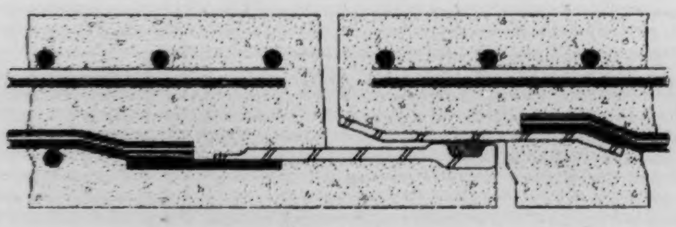


Figure 3-18 Typical cross section of steel end ring joint with spigot groove and O-ring gasket. (Figures 3-14 to 3-18 are adopted from American Concrete Pipe Association)

Applicability

This type of tunnel lining can be used in completed tunnel sections such as the north side of the W12 project, but is inappropriate for the portion for which the EPBM is to be used, as this would require jacking equipment to push the pipe. A spigot groove joint with O-ring gasket may be the best choice here to ensure water-tightness. As with pre-cast segmental concrete, however, the use of pre-cast concrete pipe necessitates specialized manufacturing techniques to ensure the required quality and precision of installation. Granted that it may not be economical to fill some 25 cm of annular space with concrete, we note that leakage in joints is not expected to be a major issue here unlike with pre-cast segmental concrete lining.

Chapter 4

Evaluation of tunnel lining systems using performance indicators

4.1 Methodology for evaluation of tunnel lining systems

Decisions should be made taking into account all available information and with the goal of reaching the optimal solution in terms of a life-cycle perspective (Rostam at al, 2004). Design of the tunnel structure progresses within those parameters determined by the geological conditions, and these parameters may result in different consequences in terms of construction and maintenance cost, durability, risk, and environmental effect. However, during the decision-making process, to consider all of the detailed information pertaining to each structure is a difficult and often time-consuming task. In fact, in many cases decisions need to be made according to a strict and tight timeline. The objective of this chapter is to introduce performance indicators for evaluation and a kind of decision support system to be applied during the preliminary design stage of the project. The purpose of performance indicators is to develop an evaluation framework which considers the maintenance and construction stages of tunnel lining systems by surveying experts' opinions. These procedures will identify more feasible alternatives for this unique tunnel project and will provide a range of alternatives related to maintenance which will aid in informing the initial design. In general, most evaluations have been informed by experts' opinion, coupled with historical and field data. Evaluation during the early stages of design is critical as it will have an effect on the design and construction process and final production. Decision makers and engineers may need transparent and objective performance requirements in order to properly evaluate and select the best alternative. As it stands, three designated alternatives are already determined to have satisfied the design criteria as pressurized tunnel lining systems. In order to evaluate the performance of each tunnel lining system through use of these indicators, we adopt the process exhibited in Figure 4-1.

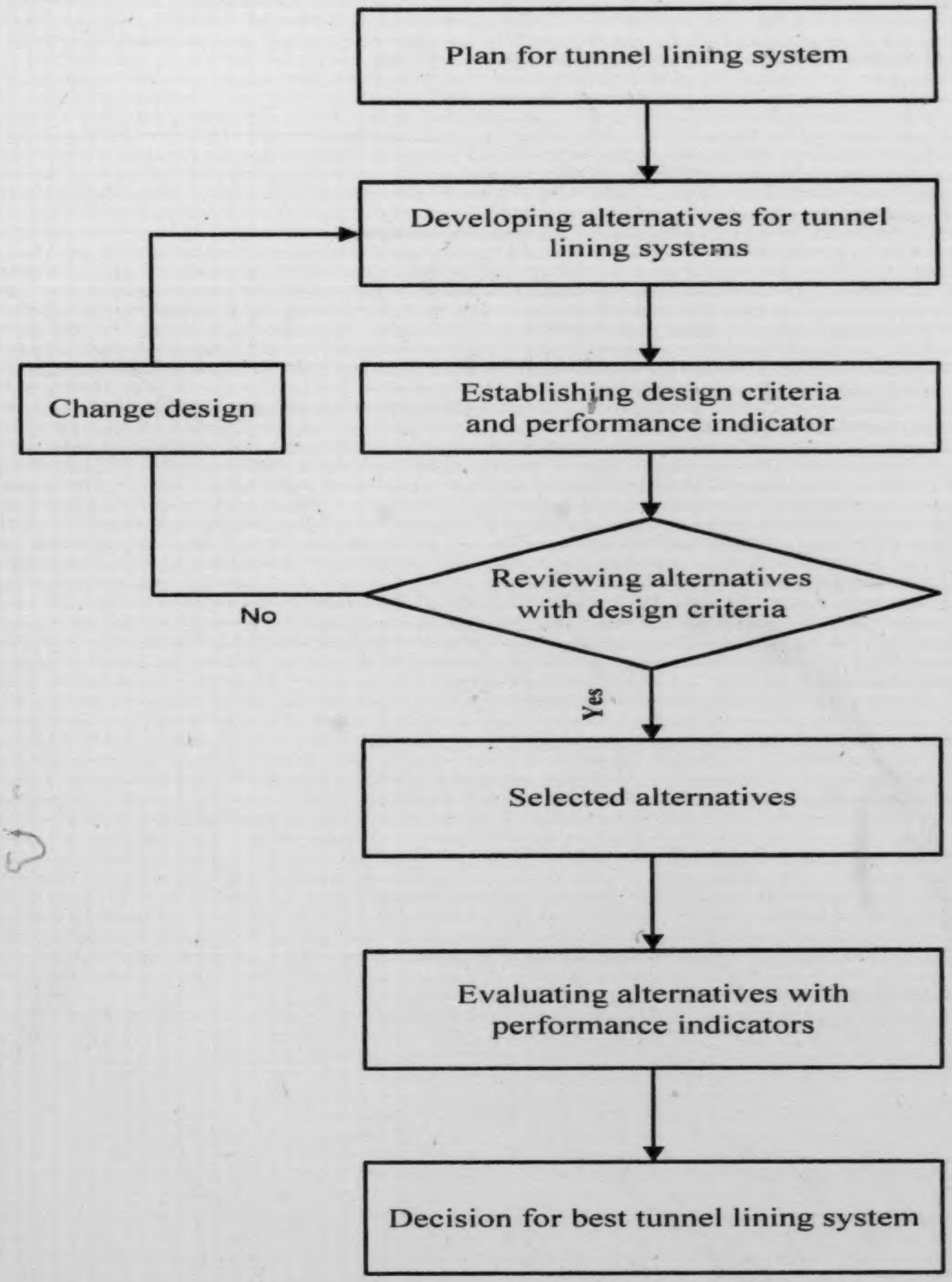

Plan for tunnel lining system
Developing alternatives for tunnel lining systems
Change design
Establishing design criteria and performance indicator
Reviewing alternatives with design criteria
No
Yes
Selected alternatives
Evaluating alternatives with performance indicators
Decision for best tunnel lining system

Figure 4-1 Evaluation flow of tunnel lining systems using performance indicators

4.2 Development of performance indicators

Indicators have often proven a powerful tool for simplifying complex situations (Yverås, 2002). This resource may be available as a decision support system in some circumstances or as a tool for evaluating alternatives in others. Performance indicators generally provide a simplified picture of reality, but are reliable enough to inform and advance decision making.

Once a method for evaluating tunnel liners has been established, tunnel lining systems are assessed using these indicators. In fact, a number of such decision making systems as analytic hierarchy process, fuzzy theory, and simulation have been used in the construction field, although these systems typically become complicated in their application. The indicators are usually used for measuring the performance of an education or economic program. In some ways, performance indicators and design criteria appear to be similar, but design criteria outline the minimum requirements of the facility, whereas performance indicators are evaluated in terms of the optimum performance of the facility. The development of performance indicators is a brainstorming process in which several experts suggest and refine the initial indicators chosen to measure or assess the alternatives. Performance criteria are divided into two broad categories for convenience of evaluating: maintenance stage and construction stage. This taxonomy will reveal useful information which will have a considerable effect on the final selection of a tunnel lining system. One should note that project cost is mainly concerned with the cost of the materials, labour and equipment needed to complete project. The selection between alternative systems, on the other hand, should encompass the costs associated with operation or maintenance. Indeed, design reviews may reduce the construction cost but increase the long-term operation costs, which are usually considerably higher than construction costs.

4.3 The process of evaluation for tunnel lining systems

Figure 4-1 presents the process of evaluation for tunnel lining systems based on performance indicators. The first step is to develop a field of alternatives and ensure that they meet the design criteria. In the case of unbolted pre-cast segmental concrete lining, further study was terminated by virtue of the lack of water-tightness. Weighting factors for the performance indicators were selected to reflect the relative importance of different categories and factors. Responses to performance indicators were quantified using a numerical rating scale, with weighting factors applied to the scores for each category. This evaluation resulted in a total score for each alternative, and generated a quantitative comparison of the alternatives.

Table 4-1 shows the rating scheme and the verbal compliant numerical rating scale used for assessment:

< Verbal compliant numerical rating scale >

- Most excellent performance: 10
- Good performance: 8
- Moderate performance: 6
- Unpredictable performance: 4
- Some deficient performance: 2

If needed, the rating scale can be used between the specified intervals (1, 3, 5, 7, 9)

Table 4-1 The contents and rating scheme

Performance Indicators	Contents and rating scheme
Maintenance stage	The maintenance and operation of pressurized sewer tunnel
Long term service life (Durability)	This includes such factors as durability and reliability. A higher rating reflects better performance.
Maximum prevention of Leakage	This assesses the leakage of internal or external water. Less leakage merits a higher rating.
Maintenance cost	This includes inspection, repair, and cleaning. Minimal cost merits a higher rating.
Quality (Joint, liner smoothness)	This focuses on such aesthetic aspects as surface smoothness, joints, spalling, and porosity of material. The better quality procures a higher rating.
Future plan compliance	This is related to usability and development of new technology in Edmonton. The higher projected usability corresponds to a higher rating.
Construction stage	
Constructability	This includes compatibility with tunneling machine as well as lining construction. Better constructability merits a higher rating.
Cost and time (Efficiency)	This includes construction cost and time. Here cost and time, efficiency, correspond to a higher rating.
Risk	This includes the probability and mitigation cost of risk. A higher rating reflects lower probability and mitigation costs.
Experience and Technique	This is related to labor experience and technique. Better practitioner familiarity with the work merits a higher rating. (If a new skill is required, the rating will be low)

4.3.1 Maintenance stage

The performance evaluation for operation and maintenance must be reviewed by means of several important criteria based on the intended function of the tunnel. These should be established based on expert knowledge and existing data. It is no small task, it should be noted, to translate these indicators into numerical figures to be used for assessment. In this research, the indicators are established by City project managers and engineers from consultants. Categories include long-term service life (durability), maximum prevention of leakage, maintenance cost, quality, and future plan compliance, and each criterion is weighted by each expert.

During the surveying, the tunnel lining system should be conceived of as a pressurized tunnel and transport sewer. As such, durability criterion has one of the strongest influences on design and construction.

In the past, maintenance concerns did not figure prominently in the decision making process, as planners often lacked proper data pertaining to maintenance costs, but cutting edge science and technology have fundamentally altered the process of decision-making. Many existing facilities have been surveyed and tested for durability and cost-effectiveness with respect to total life-cycle cost.

Long term service life (Durability)

The most difficult and controversial aspect of maintenance cost assessment for any drainage structure may lie in establishing a service life for each of the various tunnel lining systems. Service life is a function of materials, installation environment, and the effect of additional measures taken to protect the tunnel lining from deterioration.

Liners for sewer projects should provide vital services suited to the given conditions, and are intended to remain in service for as long as their maintenance can be kept up in a practical and economical way. All types of lining systems will be subject to decay and attack from both internal and external environmental conditions. The materials used in construction, method, and required design criteria should be taken into account in the design. Furthermore, special considerations may be in order for sewer delivery tunnels as necessitated by aspects of the environment. In the case of segmental linings, for example, loose fixings in bolts and gaskets will affect the long-term service life.

In addition, tunnel lining may be exposed to varied aggressive environments, and the factors that generally influence the durability of the concrete are as follows:

- operational environment
- cover to the embedded reinforcement bars
- concrete strength, and type of cement, aggregate and admixture
- water/cement ratio
- permeability, porosity, and cracks of concrete
- workmanship

The Ontario Concrete Pipe Association and ASTM standards require a minimum concrete compressive strength of between 28 and 41 MPa in concrete pipe, a value related to structural aspects and *not* durability considerations. Concrete strength is decided by such factors as available aggregates, cement/water ratio, curing procedures, and other manufacturing processes. Higher compressive strengths require an overall high quality of mixing materials, low permeability, and greater resistance to weathering and corrosion.

Service life, alternatively, is concerned with durability of the materials used. A durable material should satisfy service conditions and weathering and chemical reaction. Durability of in the materials involved in the construction of the tunnel lining is integral to performance throughout the tunnel's service life. Figure 4-2 shows that for a concrete pipe on a slope of 2.5 % and with a pH reading of 4.0, the service life will be about 70 years.

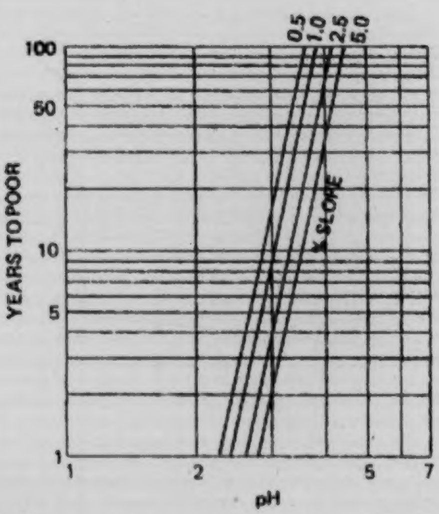


Figure 4-2 Concrete pipe service life (Adopted from the Canadian Concrete Pipe Association)

The concrete density of pipe ranges from 2150 to 2650 kg/m (Ontario concrete pipe association), and the higher densities require denser consolidation of the concrete as well as heavier aggregates. Absorption is related to the porosity of the structure, and should be considered in terms of the durability of the concrete. Furthermore, the absorption characteristics of the aggregates and the inherent characteristics of the manufacturing process affect absorption. Even though an increase in the cement content results in lower absorption, higher compressive strength, and stronger resistance to weathering in certain chemical environments, it also increases the probability of shrinkage cracking. The City of Edmonton has used type-50 cement for the sewer pipe in order to provide resistance from sulfate attack.

Aggregates should provide compatibility with a particular manufacturing process in order to achieve optimum concrete strength and control permeability. The harder and denser aggregates produce the greater abrasion resistance concrete.

Minimum cover over the reinforcing steel in concrete is specified depending on the buried condition, and should be seen as a balance between structural efficiency, durability, and cost. A thicker cover may provide longer durability against a diversity of aggressive conditions, but this design could increase significantly construction costs. The de-icing chemical agents used on bridge decks and highways in winter are the most common causes of chloride corrosion of concrete reinforcement. Chloride corrosion can occur in low quality concrete of high permeability and porosity. Durability can be maximized under such severe conditions through thicker coverage, using high quality concrete with low permeability, and without cracks or voids.

Admixtures such as accelerators, air-entraining agents, and water-reducing agents are used for a number of reasons. For instance, air entrainment agents can be mobilized in wet-cast pipe to bolster freeze-thaw and weathering resistance. Fly ash which may aid in flow is also used to provide adequate workability and pumpability.

Maximum prevention of Leakage

The influence of water on tunnel linings can be evaluated according to three general conditions: (1) control of leakage, (2) support of external water pressure, and (3) confinement of internal water pressure (Guidelines for tunnel lining design, 1984).

For the W12 project, it happens that internal water pressure is higher than external water pressure. The higher internal pressure may cause the tunnel lining to expand against the surrounding ground until the internal forces are equal to external loads, at which point tensile stresses in the tunnel lining should be introduced. This expansion may cause a cracking or loosening of bolts in the lining and result in water leakage. Just in case cast-in-place concrete is used as a final lining for the W12 project, it should be designed as reinforced concrete. In case pre-cast segmental concrete is used, the structure should be gasketed and bolted and inspected for leakage at the designated joints according to the specifications. Control of concrete cracks and leakage during curing of fresh concrete or transportation of segments may be a major issue affecting the design and determining the extent of steel reinforcement.

The primary purpose of crack control is to minimize the *maximum* crack width, as reinforced concrete design is based on a tolerable crack width value. Cracking in the concrete will be unavoidable since the concrete is relatively weak and brittle. Cracking of the pipe to the standard 0.3 mm width crack has not been found to be deleterious (Ontario concrete pipe association), and the City of Edmonton further designates 0.3 mm as suitable crack width for general sewer pipe. Cracks which completely penetrate the concrete wall are uncommon, and most cracks only reach to the first level of reinforcement steel, and tend to be widest at the surface. Having said that, it is clear that the thickness of concrete coverage is intrinsic to crack control. However, although tensile members with more than one reinforcement bar are considered, the actual concrete cover is not the most relevant variable (ACI manual of concrete practice, 224.2R-5, 2005). Instead, an effective concrete cover should be defined as a function of the reinforcement spacing as well as the concrete cover measured to the center of the reinforcement. Larger diameter pipe with coverage greater than 25 mm may show cracks exceeding 0.3 mm in width (Ontario Concrete Pipe Association). Pipe with crack widths of less than 0.3 mm may have the same durability as non-cracked concrete pipe. The ACI manual of concrete practice outlines reasonable crack widths in reinforced concrete under service loads in the following table:

Table 4-2 Reasonable crack widths and reinforced concrete under service loads (ACI 224R-19, 2005)

Exposure condition	Crack width	
	in.	Mm
Dry air or protective membrane	0.016	0.41
Humidity, moisture air, soil	0.012	0.30
Deicing chemicals	0.007	0.18
Seawater and seawater spray, wetting and drying	0.006	0.15
Water-retaining structures	0.004	0.10

The 0.3 mm crack criterion should not be regarded as conservative for pressurized tunnel lining when compared to the water-retaining structures mentioned in table 4-2. In reality, structural engineers suggest that the crack width be restricted to 0.1 mm in order to ensure leakage protection for cast-in-place reinforcement concrete lining of W12 project. It requires an additional 40% reinforcement as well as double layers of installing reinforcement bars, compared with that for a 0.3mm crack width.

Quality in tunnel lining

Quality can be defined as meeting the requirements outlined in the given specifications. One should note that although all three tunnel lining systems may satisfy the design criteria and specifications, final production will vary between the different systems based on construction or fabrication conditions. For instance, such factors related to smoothness of surface as bolt pockets may cause head loss, and joints and porosity of concrete may have an effect on leakage control. It is general practice to install transverse construction joints to simplify construction and decrease shrinkage cracks in cast-in-place concrete tunnel linings. The location and spacing of the transverse joints are normally based on the limitations of concrete pouring and steel forming. Determining a length of steel forms that provides the optimal combination of ease of handling and economy is critical. This length is generally set at between 6 and 12 m, with the transverse joints being equipped with water stops. Transverse expansion joints are not usually required as there will be no significant temperature changes in sewer systems. The longitudinal joints which divide the tunnel lining cross-section into two or more parts are designated as either invert, wall, or arch. The location of the longitudinal joint depends on the cross section and the procedure of placing concrete. Again, if water-tightness is a concern, water stops should be installed.

The joint designs of pre-cast segmental concrete linings vary depending on a project's geological conditions, construction processes, and tunneling machinery used. The size and number of segments in a ring also depend on the type of tunnel machine and support equipment. A bolted and gasketed segmental concrete lining should be used if water-tightness is required.

Pre-cast concrete pipe joints are required for such functions as resistance to exfiltration of sewage or storm water, like other linings. O-ring gaskets are generally used where specified seepage or infiltration requirements are resisted, and are suited to withstand internal or external pressures, a trait valuable to the W12 project. In terms of the loss of friction which occurs in tunnels, Manning's roughness coefficient, n, friction factor varies depending on the type of tunnel lining used (Department of the Army U.S. Army Corps of Engineers, 1997):

• Lined with cast-in-place concrete	n=0.013
• Lined with pre-cast segmental concrete	n=0.016
• Lined with steel mortar coat	n=0.014

- Lined with steel (diameter >3m) n=0.013
- Lined with steel (diameter <3m) n=0.012

In the case of pre-cast concrete pipe, the coefficient commonly applied is n=0.012 (Caltrans, 2006).

Future plan compliance

The City of Edmonton must meet the demand for infrastructure triggered by rapid population and economic growth. In view of these trends, the City has made plans to construct approximately 20 km of sewer tunnel, for instance. Some of these tunnels may be pressurized like the W12 project, and the choice of tunnel lining system may be affected by new technologies, new material and labor trends, as well as cost and durability. One of the main tendencies in the construction field is that work requiring concentration of labor has been replaced by mechanized methodologies. New material and construction methods can not only decrease construction time while increasing service life, but they promise to reduce costs and risks.

4.3.2 Construction stage

A number of factors at this stage may affect the evaluation of tunnel lining systems, such as cost and time, risk and experience, and technique.

Constructability

The Construction Industry Institute (CII) at the University of Texas in Austin defines constructability as "the optimum use of construction knowledge and experience in planning, design, procurement and field operations to achieve the overall project objectives". Based on this concept, experienced construction experts need to be involved with the project from the beginning stages in order to ensure which is the most feasible and practical tunnel lining choice for that project. It is common that constructability should be referred to as a design consideration and in the application of new techniques adopted to achieve optimum results.

Constructability in tunneling should be assessed in terms of construction methods, costs, material transport, time, sequence of excavation, muck disposal, groundwater conditions, and environmental or serviceability requirements. The tunneling method used and the timing of support may influence the final liner as well as ground movement. For pre-cast segmental concrete, loads associated with handling, placement, and thrust force are often more critical than the ground loads on segmental lining, so these forces should be considered in design. With respect to the cast-in-place concrete lining, a two-pass lining system is used and a primary liner required, consisting of ribs and lagging for most City projects. Steel rib and lagging is relatively flexible and may deform considerably to redistribute uneven ground pressures. The concrete lining should be installed after this redistribution of ground pressures has stabilized. Although the City's EPBM is equipped to install steel rib and lagging and to erect segmental concrete, steel rib and lagging does not facilitate grouting for the supporting tunnel face on unstable ground. As with the cast-in-place reinforced concrete lining system, after the initial lining has been supported using steel rib and lagging, the pre-cast concrete pipe is installed inside the tunnel using a pipe carrier. The annular space is anticipated to be around 25 cm and filled with concrete. In the case of pre-cast segment or pipe, constructability may also decide the dimensions and type of lining selected. Factors such as handling, jacking force from the TBM, and joint specifications will all impose certain conditions on the dimensions of the pre-cast segment concrete.

Cast-in-place reinforced concrete lining requires the labor-intensive installation of reinforcement bars in narrow space and, as a consequence, increases the total time and cost. Construction considerations to be considered during the installation of the cast-in-place reinforced concrete lining are as follows:

(1) *Installation of reinforcement bars*: All reinforcement bars should be accurately placed, securely fixed, and adequately maintained in their positions, with particular consideration given to concrete coverage.

(2) *Form-work*: Form-work is precisely defined in terms of dimension, erected, and securely tied up to prevent displacement of the concrete. Prior to pouring, the specified reinforcement cover should be checked.

(3) *Placing of concrete*: Concrete placement must continue uninterrupted until the structure is filled over the entire length of the formwork. Pumping equipment should have an adequate capacity and be able to delivery concrete in a continuous

flow. For compacting concrete, formwork self-vibrators are generally utilized, with the location and operation of the vibrators carefully coordinated with the withdrawal of the discharge line.

(4) *Curing*: Periods of curing must be regulated according to the given specifications.

Cost and time (Efficiency)

The feasibility of the construction process is assessed during design in order to identify any effects on the structural components or total construction cost and time. We can expect that the choice of tunneling machine may have an effect on the evaluation of tunnel lining systems. In general, the City of Edmonton has used open-faced or shielded TBMs and has installed either rib and lagging as an initial support or pre-cast segmental concrete as a permanent lining. That said, the W12 tunnel is obscured by the fact that two types of boring machines are to be used for excavation while the same tunnel lining system may be applied throughout. Cost and time will not easily be accounted for until a simulation or detailed design for each tunnel lining system has been executed. For the most part, cast-in-place reinforced concrete lining systems will be expected to require the longest completion time.

Risks during construction

One of the most important concerns in tunnel construction is risk management. Risk analysis must be mandatory, with a purpose of identifying and mitigating any issues or events that may affect the objective of the project, without limiting safety and in keeping with the project budget. Risk analysis is generally qualitative while construction method and design are developed risk quantifications carried out by experienced and knowledgeable experts. During the design review, probability and mitigation methods are utilized in order to identify the impact of risk. The risks linked to the tunnel lining are also related to geological and groundwater condition, face support methods, and the choice of excavating machine, as well as the construction of lining itself.

The results generated by geotechnical investigations equip project planners to anticipate groundwater seepage during construction, especially from residual coal seams and former coal mines. Methods for handling this flow depend on the amount, of flow as quantified by rates and volume, duration of the flow, and location of groundwater inflows, and these values may contribute to the high risk of lining installation. The first Risk Analysis Workshop for the W12 project was facilitated by SMA Consulting on September 20, 2003 with a second held on May 28, 2004. The risk analysis determined a considerable uncertainty associated with the ground conditions and hydraulic issues, and that these may significantly influence construction.

The risk factors shown in table 4-3 were considered to be critical and should be mitigated early on in the design phase.

Table 4-3 Critical risk factors (Adopted from Preliminary design report, 2005)

Risk factor rated "critical"	Suggestion for mitigation	Status
Air released in the drop shaft will impede drop filling and could result in trapped air in the syphon. This may cause irregular or unsteady flow with associated dynamic forces, reduced capacity, and may lead to upstream surcharging.	Review of other drop shaft designs leading to preliminary design. Quantify problem, develop alternatives, consider need for a physical model, slope syphon up to the south to allow air release at both ends.	Design analysis has quantified the problem and based on hydraulic, modeling and gate operation to risk is considered minimal.
Undetected voids next to shaft liner or tunnel liner may create a shaft or tunnel that is incapable of handling the pressure it was designed for.	Design liner to take load (i.e. design it to be thicker). Alternatively, carry out ground penetration radar etc. to detect voids after primary liner is in place.	Geotechnical Investigation indicates likelihood of voids 50/50. Impact is negligible as liner is to be designed to span voids. Also another test hole will be drilled during design at shaft location.
Vertical alignment may be designed deeper than required (to reduce construction failures) which may add to costs and delivery schedule.	The depth is planned in such a way as to avoid coal mines and in light of safety issues. We need to consider life cycle costs (pumps, maintenance, etc.) and the impact to construction costs arising from the depth of an 80 m shaft.	Alignment has been set to mitigate risk.
Large volumes and rates of flow during extreme events may overpower the conveyance capacity or structural strength of the downstream system or operation aspect at Gold Bar Waste Water Treatment Plant (GBWWTP).	Keep syphon full to minimize impacts. Flow control/surge suppression at McNally.	A full syphon operation condition has been adopted for the design.

Experience and technique

Engineering judgments based on human knowledge and experience have always played an important role in tunneling design and construction. The "experience and technique" indicator is related to human factors which may affect the productivity and reliability of the construction. Experience coupled with the mobilization of piled techniques may serve to reduce risk and error in construction as well as the total construction time involved. According to Sowers research (1993), some 88% of construction failures were due to "human shortcomings": particularly, not understanding or properly employing contemporary technology; interestingly, only 12% of the failures were due to technological shortcomings. Sowers offered several suggestions that could help to minimize human error, including continuing education, better enforcement of engineering registration laws, an increase in awareness among engineers of their limitations in making decisions, improvement of communications, and the incorporation of practicing engineers' expertise as new technique develops.

Compared to other sewer tunnel projects constructed by the City of Edmonton, the W12 project is quite unique. Therefore, engineering judgment needs to meet the standards outlined in the results of the risk analysis and defined by the unique conditions of the project; it also needs to incorporate the selection of workable alternatives and improvement of constructability. In light of the noted geotechnical complications involved in the construction process, experienced engineers should review the actual performance of the equipment and other resources during construction, ready to make modifications easily and efficiently if required. These decisions are informed by the experience and technique of expert engineers, although it is often difficult to account for or quantify numerically the contributions of expert engineers in this area.

4.4 The result of evaluation for tunnel lining systems

The evaluation of tunnel lining systems based on performance indicators was executed via personal meetings and emailing. Participants included experts either from the City or from engineering firms involved with the W12 project. Moreover, each of the expert participants offered a wealth of knowledge to do with tunnel construction and understood well the geological and environmental conditions involved with geotechnical engineering and construction in Edmonton. Experts treated the maintenance stage as of much greater importance than the construction stage, and the long-term service life was regarded as the weightiest factor affecting the evaluation of tunnel lining systems. The weighting of the various performance indicators is shown in table 4-4, in compliance with infrastructure industry standards.

Table 4-4 Weight of performance indicators

Performance indicators	Weight		Degree of importance
Maintenance stage	**0.625(1)**	Weight*(1)	
Long-term service life (Durability)	0.425	0.266	1
Maximum prevention of leakage	0.175	0.109	4
Maintenance cost	0.238	0.148	2
Quality (joints, liner smoothness)	0.075	0.047	9
Future plan compliance	0.088	0.055	8
Construction stage	**0.375(2)**	Weight*(2)	
Constructability	0.225	0.084	6
Cost and time (Efficiency)	0.338	0.127	3
Risk	0.288	0.108	5
Experience and technique	0.150	0.056	7

In the maintenance stage, we find that long-term service life (durability) is the most important indicator while maintenance cost is ranked second. One can safely conclude that good design ensures low maintenance, and maintenance costs must be considered by designers. This means that a durable lining must be able to perform satisfactorily during its expected design life.

At the construction stage, cost and time have the greatest influence on the evaluation. In practice, economic factors are relatively easy to quantify, and these are the easiest indicator for which to gather exact information. Figures 4-3 and 4-4 below show the results of evaluations for tunnel lining systems at both the maintenance stage and the construction stage.

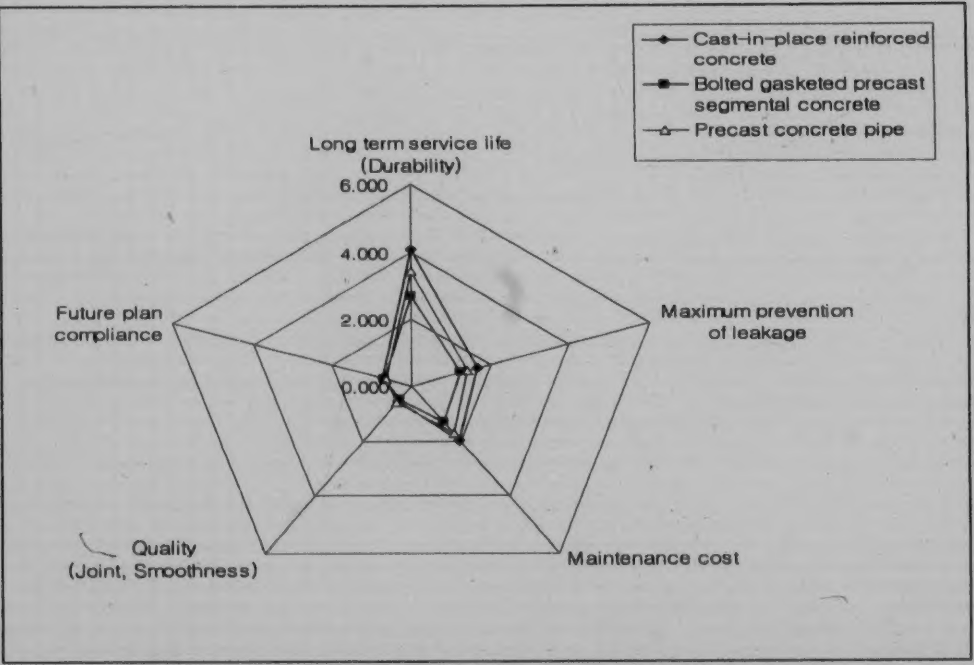


Figure 4-3 Performance characteristics of tunnel lining systems in maintenance stage

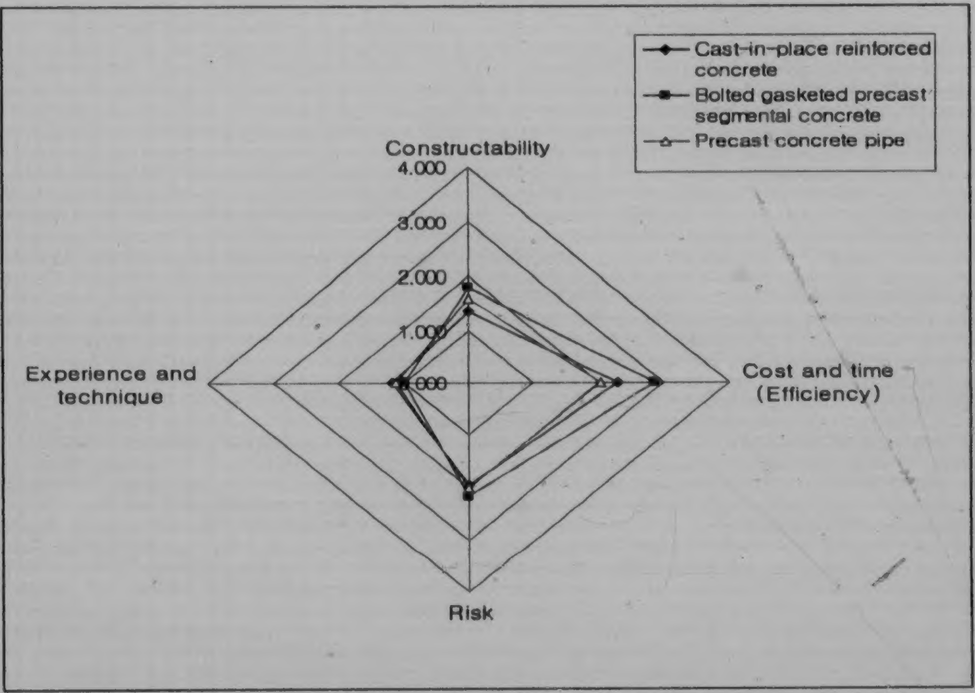


Figure 4-4 Performance characteristics of tunnel lining systems during the construction stage

In summary, the performance indicators play a vital role in deciding which factors will have a significant effect on the performance of the tunnel lining as well as in selecting the appropriate tunnel lining system for handling internal and external pressures. Among the three types of tunnel lining systems, shown in table 4-5, cast-in-place reinforced concrete lining is determined to be the most suitable lining for pressurized tunnel, and pre-cast concrete pipe the second. Although bolted and gasketed pre-cast segmental concrete lining is the most suitable tunnel lining system at the construction stage, it shows poorly in terms of maintenance and durability. With its many joints and bolt pockets, its poor performance assessment for the maintenance stage (especially regarding durability and leakage prevention) contributes to a low overall rating.

Now, in view of these evaluation results, we conclude that long-term durability is the most significant performance indicator pertaining to concrete liners. Joints may well be the weakest points in both pre-cast segmental and pre-cast pipe systems. Experts are justifiably concerned about the water-tightness of both liners as the joints eventually loosen following construction. At the construction stage, while cast-in-place reinforced concrete is given the lowest rating, bolted and gasketed segmental concrete lining scores the highest. This is because pre-cast segmental concrete is the most appropriate to TBM tunneling while, in the case of cast-in-place reinforced concrete, installation of reinforcement bars (in narrow spaces) is rather difficult. Consequently, it is possible that in view of maintenance concerns, pre-cast segmental concrete should be improved by ensuring the reliability of joints, while in terms of construction, cast-in-place reinforced concrete should be improved particularly easing the process of installing reinforcement bars. A number of studies have suggested that minimizing cracking and permeability is of utmost importance to maximizing durability. In this respect, tunnel lining systems can be improved by using fiber reinforcement concrete regardless of the lining type.

Table 4-5 Evaluation of tunnel lining systems using performance indicators

Performance indicators	Weight	Cast-in-place reinforced concrete		Bolted gasketed pre-cast segmental concrete		Pre-cast concrete Pipe	
		Rating	Weight * Rating	Rating	Weight * Rating	Rating	Weight * Rating
Maintenance stage	**0.625**		**5.590**		**4.012**		**4.859**
- Long term service life(durability)	0.425	9.500	4.038	6.250	2.656	8.000	3.400
- Maximum prevention of leakage	0.175	9.500	1.663	7.250	1.269	8.000	1.400
- Maintenance cost	0.238	8.250	1.959	5.500	1.306	7.250	1.722
- Quality(Joint, Smoothness)	0.075	7.500	0.563	6.500	0.488	8.250	0.619
- Future plan compliance	0.088	8.250	0.722	8.000	0.700	7.250	0.634
Small sum	1.000	43.000	8.944	33.500	6.419	38.750	7.775
Construction stage	**0.375**		**2.524**		**2.925**		**2.527**
- Constructability	0.225	6.000	1.350	8.000	1.800	7.000	1.575
- Cost and time (Efficiency)	0.338	6.750	2.278	8.500	2.869	6.000	2.025
- Risk	0.288	6.750	1.941	7.500	2.156	7.000	2.013
- Experience and technique	0.150	7.750	1.163	6.500	0.975	7.500	1.125
Small sum	1.000	27.250	6.731	30.500	7.800	27.500	6.738
Total score		**8.114**		**6.937**		**7.386**	

4.5. Application of SFRC as tunnel lining

Surveys show that long-term service life (durability) is the most important factor related to sewer tunnel lining. As a veritable volume of research intimates, fiber reinforcement can improve the tensile strength and durability of concrete. More recently, several types of fibers have been used in buildings, pavement, and tunnel linings. The W12 tunnel is pressurized and should be protected from leakage or infiltration. In this respect, fiber reinforcement will play an important role in increasing impact load and flexural strength (Altun et al, 2007 and Bischoff et al, 2003), and in controlling cracking and permeability (Rapoport et al, 2002).

4.5.1 Materials for fiber reinforced concrete

Fiber reinforcement is becoming an increasingly popular means of improving the mechanical properties in tension-weak concrete. Plain concrete has a low tensile strength and strain capacity at fracture, but these disadvantages are overcome through the addition of reinforcement bar. Many types of fibers are available for commercial and experimental use in cement-based composites, such as steel, glass, polypropylene, asbestos, and natural fibers. Typical properties of the fibers are listed in table 4-6 below:

Table 4-6 Types of fibers and properties (Mulolick et al. 2006)

Types of fibers	Specific Gravity	Tensile Strength, MPa	E, GN/m^2	Elongation at failure, %	Common V$_f$, %
Polypropylene	0.91	550-700	3.5-6.8	21	<2
Steel	7.86	400-1200	200	~3.5	<2
Glass	2.7	1200-1700	73	~3.5	4-6
Asbestos	2.55	210-2000	159	2-3	7-18
Polyester	1.4	400-600	8.4-16	11-13	~0.065
Concrete, for comparison	2.4	2-6	20-50		0

Commercially used mixes in fiber reinforced concrete (FRC) are often similar to conventional reinforcement concrete mixes. But while reinforcement bar is continuous and is installed in the structure to optimize performance, fibers are discontinuous and are distributed randomly throughout the concrete matrix. So although the fibers are distributed homogenously throughout the concrete matrix, they are also present in the compression area. The mechanical properties of FRC are influenced considerably by the fiber-aspect ratio, fiber-volume fraction, fiber type, fiber orientation and distribution, and the properties of the fiber-matrix interface.

(1) Fiber aspect ratio (l/d) is defined as the ratio of the length to the equivalent fiber diameter, and influences both the workability of the wet mix and the mechanical properties of the hardened composite (Council on tall buildings and urban habitat committee 21D). To avoid fiber balling in conventional mixing, and to provide a neat uniform distribution of fibers in the mix, a maximum aspect ratio of 100 is usually recommended.

(2) Fiber volume fractions typically used in conventional FRC range from 0.1 to 2.0%.

(3) Fiber types currently being used in concrete can be classified broadly into two types. The first type is low volume, high-elongation fibers such as acrylic, nylon and polypropylene. Here, a suitable aspect ratio will range from 50 to 100. The second type is high-modulus, high-strength fibers, such as steel, glass, and carbon.

4.5.2 The purpose of SFRC as a tunnel lining system

The bipartite purpose of the tunnel lining is to support external and internal load while preventing long-term water leakage. To design concrete lining economically while improving its durability, it is essential to increase the load-carrying capacity of the concrete while minimizing cracking and permeability. It has been well established that one of the important properties of steel fiber reinforced concrete (SFRC) is its excellent resistance to cracking and crack propagation (Mashimo et al, 2006). This feature enables fiber-reinforced concrete to increase its tensile strength, which in turn impacts strength, toughness, fatigue strength, and the ability to resist spalling.

The overall result of this may be the enhancement of the concrete's durability. In the case of the inverted syphon tunnel, the tunnel lining should have the ability to support repeatedly applied dynamic or impact loading, such as water-hammer. Variable internal pressure and water hammer cause fatigue and crack propagation of the tunnel lining so that increased toughness and crack stitching are required. According to structural analysis, in order to limit crack widths to less than 0.3 mm (a design criteria), one layer of reinforcement bar should be installed. In addition, to reduce occurrences of cracks with widths up to 0.1 - 0.2 mm requires the additional installation of reinforcement bars in the range of about 20% ~ 40%. If a greater quantity of reinforcement bars is used to arrest cracks, two layers of reinforcement should be used. The work of installing reinforcement bars will be difficult and must be more precise. Conventional reinforcement bars ought to be used to confine cracks of up to 0.1 mm crack in width, but in many cases these cracks appear because of shrinkage of the concrete as well as a lack of tensile stresses. In this respect, steel FRC is recommended for reducing crack width and increasing flexural toughness.

4.5.3 Material and mixture proportioning

The mixture proportions for SFRC depend on the requirements for a particular project in terms of structure, concrete strength, workability, pumpability, et cetera.

SFRC mixtures contain higher cement contents and higher ratios of fine-coarse aggregate than do conventional concrete mixtures, and so the mix design procedures that apply to conventional concrete may not be entirely applicable to SFRC (ACI 544.1R, 2005). In addition, to improve the workability or pumpability of higher fiber volume mixes, water reducing admixtures and, in particular, super-plasticizers are often used. The range of proportions for normal volume SFRC is proposed in Table 4-7. The second factor which has a major effect on workability is the aspect ratio (*l/d*) of the fibers. The workability decreases as the aspect ratio increases. Practically speaking, it is very difficult to achieve a uniform mix if the aspect ratio is greater than about 100. As informed by the literature and in light of an ACI value of 302.1R, a quantity of steel fiber about 0.5% by volume of concrete (40kg/m^3) is recommended.

Leung and Shapiro (1999) suggest that optimal fiber yield strength for maximum pull-out load and energy absorption will range from 635-954 MPa. Proposed steel fibers fall within the general length range of 12.7-63.5 mm and with common aspect ratios within the range 30-100. In many projects, steel fibers have been mixed without any changes to the conventional concrete. In table 4-7, the mixture of steel fiber is compared with the ACI range of proportions.

The proportions assume that the strength is 30 MPa with type 50 cement, non-air entrained, with a slump of 140 mm, and that a normal range water reducer (WRDA64) and mid-range water reducer (Daracem 18) are to be used.

Table 4-7 Mixture proportioning for SFRC

	ACI manual (ACI 544.1R, 2005)		Propose for W12 Project
Maximum-size aggregate	9.525mm (3/8 in.)	19.05mm (3/4in)	14mm
Cement, kg/m^3	356-593	297-534	340
w/c ratio	0.35-0.45	0.35-0.50	0.46
Percent of fine to coarse aggregate	45-60	45-55	42
Entrained air content (percentage)	4-8	4-6	0
Steel fiber content, Vol. percent Deformed fiber	0.4-1.0	0.3-0.8	About 0.5 (Figure 4-5)

* 1 lb/yd3=0.5933kg/m^3, 1in.=2.54cm, 1 steel fiber volume percent=132.3lb/yd3(78.5kg/m^3)

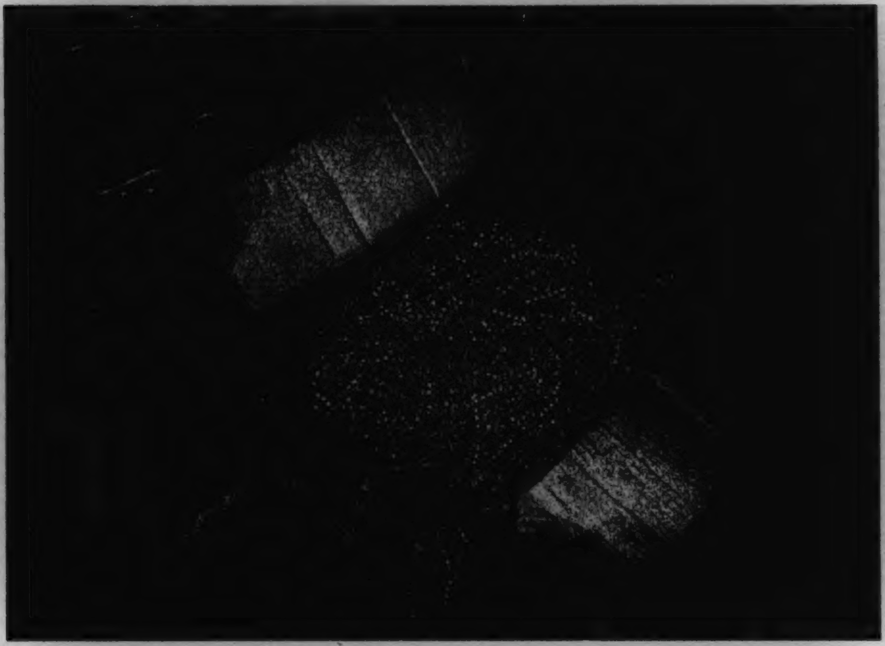


Figure 4-5 Steel fiber (L=45mm)

Mixing in concrete

For the mixing of SFRC it is essential to achieve a uniform distribution and to prevent the segregation or balling of the fibers. This balling of the fibers is related to several factors in mixing: the aspect ratio of the fibers, the volume of the fibers, the maximum size and gradation of the aggregates, and the method of adding the fibers into the mixture. As shown in figures 4-6, 4-7 and 4-8, three mixing methods have been successfully used for uniform distribution of fibers and are summarized as follows (ACI 554.1R, 2005):

(1) The first method is to add the fibers to the truck mixer after all other ingredients have been added and mixed. Steel fibers should be added to the mixer hopper at the rate of about 45 kg per minute, with the mixer rotating at full speed. The fibers should be added in a clump-free state so that the mixer blades can carry the fibers onto the mixer. As shown in figure 4-6, steel fibers are added manually by emptying the containers into the truck hopper, or via a conveyor belt or blower.

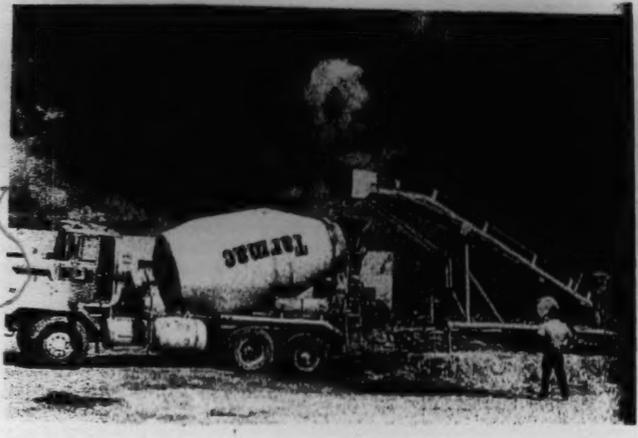


Figure 4-6 Adding steel fibers to a loaded mixer truck via a conveyor (ACI 554. 1R, 2005)

(2) The second method is to add the fibers to the aggregate stream in the batch plant before the aggregate is added to the mixer. Steel fibers can be added manually on top of the aggregates in the charging conveyor belt as shown in figure 4-7.

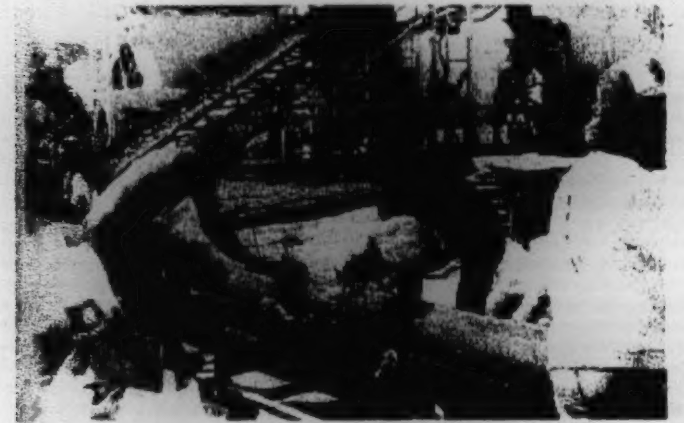


Figure 4-7 Adding steel fibers on to a charging conveyor in a batch plant (ACI 554. 1R, 2005)

(3) The third method is to add the fibers on top of the aggregates after they are weighed in the batcher. The normal flow of the aggregates out of the weigh batcher will distribute the fibers throughout the aggregates. Steel fibers can be added manually or via a conveyor as shown in figure 4-8.

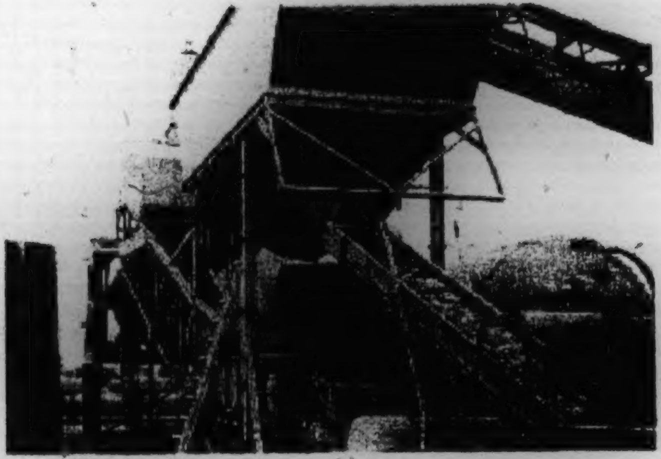


Figure 4-8 Adding steel fibers to weigh batcher via conveyor belt (ACI 554. 1R, 2005)

The first method is commonly used in Edmonton-area projects at the batch plant or on the job site prior to placement, but this method cannot always guarantee even distribution in the front of the truck mixer, since the fibers are initially placed in the rear of the truck mixer. The second and third methods are more effective in distributing fibers evenly during mixing.

In general, some SFRC mixtures are characterized by higher cement and fine aggregate content and decreasing slump, with increasing fiber content compared to conventional concrete mixes. At relatively small job sites, the first method is successfully applied, but in order to avoid clumping, the second and third alternatives are better.

4.5.4 Formwork and finishing

The process of designing and constructing the formwork should take into account the concrete weight and other loads. Typically, a normal weight SFRC with a fiber content up to 2% by volume has a density similar to that of normal weight conventional concrete, in the range of 2306 to 2403kg/m^3 (ACI 544.3R, 2005). Therefore, special consideration is not needed in formwork and finishing, except that practitioners must be careful at sharp corners due to the protrusion of steel fibers.

The transporting and placement of SFRC can be accomplished with the same conventional equipment. Pumping has been used to delivery concrete on a number of projects, and a good fiber mixture normally has proportions of sand and admixtures which make it well-suited to pumping. Although a mixture may appear stiff and unworkable, it may turn out to pump surprisingly well. Because of its composition, an SFRC mixture will move through the line without slugs and has been reported to pump more easily and with less trouble than conventional concrete (ACI 544.3R, 2005). However, there are some important points to consider about pumping SFRC. As seen Table 4-7, mixture proportioning should factor in pumpability:

- Use a pump capable of handling the volume and pressures as well as a large-diameter pipe line, preferably of at least 6 in.(150 mm)
- Avoid the use of flexible hose if possible
- Provide a screen over the pump hopper to prevent any fiber balls from entering the pipe line; about 50*75mm mesh is usually adequate
- Do not attempt to pump a fibrous mix that is too wet.

SFRC will be used in conjunction with reinforcing steel if selected in the W12 project, and so the spacing of bars should be carefully considered. The fiber length should not exceed the clear spacing between bars. SFRC can be finished with conventional equipment and no special attention is normally needed for flat-formed surfaces, although minor refinements in techniques and workmanship such as for chamfers and rounds may be required. Overworking should not occur during finishing, as it may bring excessive fines to the surface and result in cracking, (a condition which normally appears following the curing period). Curing and protection of the newly-poured SFRC should be carried out in the same way as for conventional concrete.

4.5.5 Prevention of fiber balling

Fiber balling commonly occurs before the fibers get into the mixture. Once the fibers have been mixed ball-free, they almost always remain in that condition. This means that if clogs form, this is likely due to the fact that fibers were added in such a way that they fell on each other and stacked up in the mixer or on the belt. Such a phenomenon normally occurs when the fibers are being added too rapidly at some point in the procedure. To prevent balling, the fibers should not be allowed to pile up or slide down the vanes of a partially filled drum. Other causes of balling may be either the addition of too many fibers to a mixture, (generally more than 2% by volume or a high aspect ratio).

4.5.6 Pumpability of SFRC

SFRC can typically be pumped without great difficulty. While the addition of steel fibers can affect viscosity and flow characteristics, for the most part they do not have an adverse effect on the pumpability of the concrete to which they are added. In the case of steel fibers, a proportion of less than 1% by volume will not affect the pumpability. The proposed mixture uses around 0.5% by fiber volume, so no problems during pumping are expected.

4.5.7 The effect of using SFRC

According to the hydraulic analysis of the W12 project, the inverted syphon should be able to support a 2 m cyclic surge coupled with the internal water pressures. Cyclic loading is critical for crack propagation and may affect the durability of the tunnel lining. As the previous chapter mentioned, taking full advantage of steel fibers, the use of steel fibers in conventional reinforced concrete tunnel linings can be very effective in equipping the tunnel to resist cyclic surges, impact loading, and fatigue performance.

In addressing the excessive groundwater and high internal water pressure associated with the W12 project, SFRC may also play a significant role in increasing durability and preventing cracking and leakage.

4.5.8 Limitation of application

At present there is no generally accepted design method to apply SFRC in lieu of conventional reinforcement bars. In Japan, although a design method based on fracture mechanics has been developed which can compute the load-carrying capacity of SFRC, this design method has not been approved and generalized.

In this chapter, a lack of available design guidelines and reference prevent a proper description of the extent to which reinforcement bars may be reduced by the use of steel fiber reinforcement. Many studies have indicated that fiber reinforcement is very effective with regard to post cracking and impact load, but such experimental results are not sufficient to use fibers as a structural component. For this thesis, based on a literature review, we may conclude that a value of around 0.5% by volume of SFRC is recommended for enhancing the durability of the tunnel lining by reducing permeability and cracking.

4.6 The proposed placing concrete method for tunnel lining

Placement of concrete for the tunnel lining generally involves the pouring of concrete into the crown of the steel form (see figure 4-9). This practice ensures a long flow path toward the final destination. It is of note, that the lateral flow of concrete between reinforcement bars may cause segregation. Interestingly, this placing method is very common in tunnel lining construction. The proposed method is intended to decrease the flowing distance of concrete as well as the vibration required to facilitate the flow. As seen in figure 4-10, the improved placement method involves the addition of ports in both sides of the steel form as well as at the crown. During the pouring of concrete into the side ports, it may be possible to delivery concrete faster by means of a single crown port, depending on the pump pressure. As a result, the proposed method can improve the productivity of concrete placement and is more effective in preventing segregation of aggregate. These effects lead to increased durability of the tunnel liner.

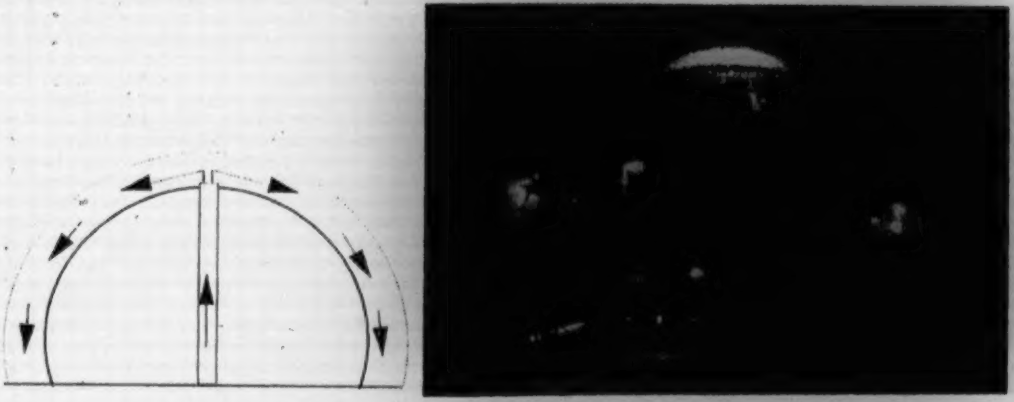


Figure 4-9 Conventional placement method in Seoul subway construction

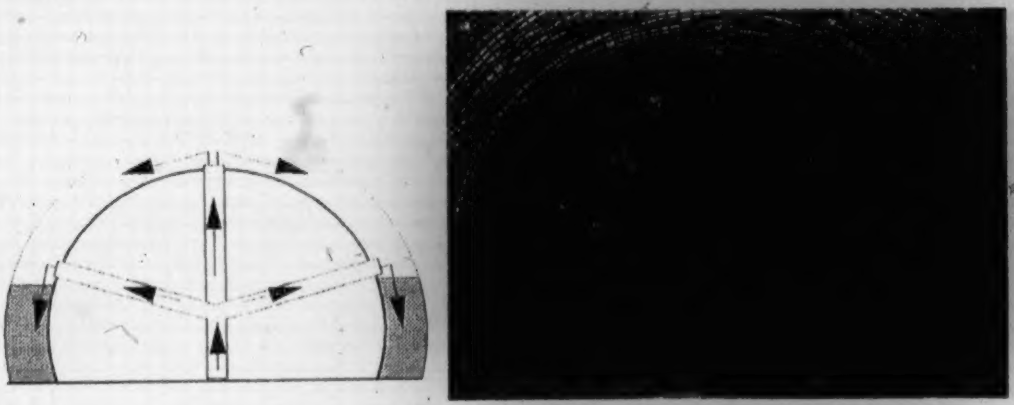


Figure 4-10 Proposed placing method in Seoul subway construction

Chapter 5

Conclusion and recommendation for future work

5.1 Research summary

This research presents an evaluation of tunnel lining systems for internal and external pressures, and assesses the feasibility of using SFRC to create a durable tunnel lining. Performance indicators are used in the evaluation of tunnel lining systems, and the result of this evaluation plays an important role in selecting a suitable tunnel lining system with respect to internal and external pressures, as well as in determining what is the most important mechanical property to consider in assessing tunnel performance. As a result of this research, cast-in-place reinforced concrete lining has been deemed to be the most applicable for pressurized tunnels overall, although for the construction stage, bolted and gasketed pre-cast segmental concrete lining receives the highest scores. Among the various performance indicators, durability of concrete is found to be the most important factor affecting tunnel lining, and is influenced by such conditions as cracking and permeability.

The feasibility of using SFRC for durable tunnel lining was confirmed based on an examination of the relevant literature as well as a concrete company's data. Although SFRC cannot replace conventional reinforcement bars as structural components, it presents a number of advantages for increasing durability. Especially as it pertains to the W12 project, steel fiber reinforcement can play a significant role in enhancing the long-term service life by reducing crackages and permeability. Indeed, in light of manufacturers' and engineers' endeavors, the notion of using fiber reinforcement as a structural component may be more plausible in the near future.

5.2 Conclusion and contribution

Many designers and planners have suffered from the pressure of having had too little time to make an important decision, despite having had a number of decision support systems at their disposal. In some respects, performance evaluation using indicators as a decision tool is very simple, but performance indicators can often suffice in considering expert experience and knowledge for evaluation. Although it is recognized that a considerable gap always exists between academic knowledge and reality, it is often time-consuming and expensive to develop complicated simulation programs in the preliminary stages of a project. By using performance indicators for evaluation, the planner and designer are enabled to identify and consider integrated expert opinions related to performance as well as details on the construction process of various facilities. The proposed evaluation process involves not only the design criteria but also performance requirements.

As a result of the evaluation of tunnel lining systems, we conclude that performance indicators play an important role in determining which factors have a significant effect on the performance of tunnel lining systems, as well as in selecting the appropriate tunnel lining system for internal and external pressure. Among the three types of tunnel lining systems, cast-in-place reinforced concrete lining is assessed as the most suitable liner for pressurized tunnels, and pre-cast concrete pipe the second. Although bolted and gasketed pre-cast segmental concrete lining is the most suitable system at construction stage, this lining has some disadvantages in terms of maintenance, as the great number of joints and bolt pockets hamper leakage prevention and durability.

As for cast-in-place reinforced concrete, it has been concluded that constructability should be improved. The use of SFRC may be a viable solution if the design concept is changed. As mentioned, it is not possible to describe to what extent the use of reinforcement bars will be reduced through the introduction of steel fiber reinforcement, as we lack a suitable design model at present. The conclusion of a number of studies is that the addition of about 0.5% steel fiber into reinforced concrete will enhance the durability of the tunnel lining by reducing permeability and crackage as well as improving ductility.

At the construction stage, cast-in-place reinforced concrete is ranked lowest, while bolted and gasketed segmental concrete lining receive the best ranking. This can be explained by the fact that pre-cast segmental concrete is the most appropriate to TBM tunneling while, in the case of cast-in-place reinforced concrete, we note that the installation of reinforcement bars, formwork, and concrete placement are especially difficult in small tunnels. Consequently, it is possible that in view of maintenance issues, pre-cast segmental concrete should be improved by ensuring the reliability of joints, whereas in view of construction concerns, cast-in-place reinforced concrete should be bettered by improving constructability, particularly in terms of the installation of reinforcement bars. A number of studies have indicated that crackage and permeability of concrete have a maximum effect on durability, and application of SFRC can increase the durability of concrete. The proposed concrete placement method can also be instrumental in enhancing the quality of concrete.

Use of the performance indicators outlined in this paper would provide a valuable practice as a kind of decision support system integrating the expert knowledge and experience of construction management. Although there is no sufficient experimental test, application of SFRC can reduce the crackage and permeability of pressurized tunnel lining and increase the long-term service life, the most significant factor among performance indicators.

5.3 Recommendation for further research

Performance indicators are used in the evaluation of tunnel lining systems and play an important role in *selecting* suitable lining systems. It is believed that each performance indicator requires a corresponding numerical evaluation criteria in order to facilitate further objective assessment of performance. In reality, it is not easy to quantify precisely long-term service life, for example, so the indicator must be assessed only with respect to expert experience and knowledge. In this respect, further research is needed to quantify these performance indicators when possible, depending on the tunnel lining systems.

As mentioned above, the most significant barrier to the application and acceptance of fiber reinforced concrete as a structural component of tunnel lining is the lack of design concepts and guidelines. To overcome this deficiency, further studies should continue to establish new structural design guidelines pertaining to fiber reinforcement.

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Appendix A - The survey for evaluating the tunnel lining systems

1. The method of survey

We personally meet or email experts for the survey and explain the geological conditions, purpose, and method of assessment for the evaluation. To supplement the email communication, we called and provided them with detailed explanations about the purpose and conditions of the survey. Six experienced experts recommended by the City project manager were selected by virtue of their extensive knowledge and understanding of the characteristics and operation conditions of the W12 project and general geological conditions associated with tunneling in the Edmonton area. Two of the experts selected failed to respond to this survey. The other four experts' opinions proved very helpful in refining the contents and rating scheme of the performance indicators. Following development of the performance indicators, the experts assessed the evaluation of tunnel lining systems for internal and external pressure using the performance indicators.

2. Purpose of the survey

Performance indicators are a measurement that describes how well an alternative scores in achieving the W12's tunnel lining objectives. The performance indicators will be employed in the selection of a suitable tunnel lining system from among three alternative systems: cast-in-place reinforced concrete, bolted and gasketed pre-cast segmental concrete, and pre-cast concrete pipe. The evaluation will render one factor to be regarded as having the greatest bearing on the performance indicators, and this information will prove imperative to tunnel lining design.

3. The condition of the tunnel lining system

The survey is to be performed in the context of the W12 project. The tunnel lining system should withstand internal and external water pressure as well as the strain of overburden load. The tunneling method should employ either TBM or EPBM technology. The geological and groundwater table conditions also refer to the result of the geotechnical investigation of the W12 project.

4. Evaluation method

Weighting factors are selected so as to reflect the relative importance of different stages and for each indicator.

- First: To settle the weight of "Maintenance stage" and "Construction stage", and then to assign a weight to each performance indicator.
- Second: To assess each alternative using the contents of the indicators and along with the verbal compliant numerical rating scale in the evaluation form.

5. Verbal compliant numerical rating scale

- Most excellent performance: 10
- Good performance: 8
- Moderate performance: 6
- Unpredictable performance: 4
- Some deficient performance: 2

If needed, the rating scale can be used between the specified intervals (1, 3, 5, 7, 9).

6. The content and rating scheme

Performance Indicators		Contents and rating scheme
Maintenance stage		The maintenance and operation of pressurized sewer tunnel
	Long term service life (Durability)	This includes such factors as durability and reliability. A higher rating reflects better performance.
	Maximum prevention of Leakage	This assesses the leakage of internal or external water. Less leakage merits a higher rating.
	Maintenance cost	This includes inspection, repair, and cleaning. Minimal cost merits a higher rating.
	Quality (Joint, smoothness)	This focuses on such aesthetic aspects as surface smoothness, joints, spalling, and porosity of material. The better quality procures a higher rating.
	Future plan Compliance	This is related to usability and development of new technology in Edmonton. The higher projected usability corresponds to a higher rating.
Construction stage		
	Constructability	This includes compatibility with tunneling machine as well as lining construction. Better constructability merits a higher rating.
	Cost and time (Efficiency)	This includes construction cost and time. Here cost and time, efficiency, correspond to a higher rating.
	Risk during construction	This includes the probability and mitigation cost of risk. A higher rating reflects lower probability and mitigation costs.
	Experience and Technique	This is related to labor experience and technique. Better practitioner familiarity with the work merits a higher rating. (If a new skill is required, the rating will be low)

7. Evaluation form

Performance Indicators	Weight	Cast-in-place reinforced concrete	Bolted and gasketed pre-cast segmental concrete	Pre-cast concrete pipe
Maintenance stage				
Long term service life (Durability)				
Maximum prevention of Leakage				
Maintenance cost				
Quality (Joint, smoothness)				
Future plan compliance				
Construction stage				
Constructability				
Cost and time (Efficiency)				
Risk				
Experience and Technique				

Appendix B - The result of survey

1. The weighting factor

Performance indicators	Weighting factor					
	Expert A	Expert B	Expert C	Expert D	Average	Degree of importance
Maintenance stage	0.8	0.7	0.6	0.4	0.625 (E)	
Long term service life(Durability)	0.5	0.5	0.35	0.35	0.425 (F)	0.266 (G)
Maximum prevention of leakage	0.2	0.15	0.2	0.15	0.175	0.109
Maintenance cost	0.2	0.2	0.3	0.25	0.238	0.148
Quality (Joint, smoothness)	0.05	0.05	0.05	0.15	0.075	0.047
Future plan compliance	0.05	0.1	0.1	0.1	0.088	0.055
Construction stage	0.2	0.3	0.4	0.6	0.375	
Constructability	0.3	0.3	0.2	0.1	0.225	0.084
Cost and time (Efficiency)	0.2	0.3	0.1	0.75	0.338	0.127
Risk	0.3	0.3	0.45	0.1	0.288	0.108
Experience and technique	0.2	0.1	0.25	0.05	0.150	0.056

* Average=Expert (A+B+C+D)/4

* Degree of importance (G)= E*F

2. The numerical rating of cast-in-place reinforced concrete lining

Performance indicators	Numerical rating				
	Expert A	Expert B	Expert C	Expert D	Application (Average)
Maintenance stage					
Long term service life(Durability)	9	10	10	9	9.500
Maximum prevention of leakage	9	10	10	9	9.500
Maintenance cost	8	8	8	9	8.250
Quality (Joint, smoothness)	5	6	10	9	7.500
Future plan compliance	8	10	6	9	8.250
Construction stage					
Constructability	6	6	6	6	6.000
Cost and time (Efficiency)	7	6	6	8	6.750
Risk	7	8	6	6	6.750
Experience and technique	8	7	8	8	7.750

* Average=Expert (A+B+C+D)/4

3. The numerical rating of pre-cast segmental concrete lining

Performance indicators	Numerical rating				
	Expert A	Expert B	Expert C	Expert D	Application (Average)
Maintenance stage					
Long term service life(Durability)	4	8	6	7	6.250
Maximum prevention of leakage	6	10	6	7	7.250
Maintenance cost	4	6	6	6	5.500
Quality (Joint, smoothness)	6	8	6	6	6.500
Future plan compliance	8	10	8	6	8.000
Construction stage					
Constructability	8	8	10	6	8.000
Cost and time (Efficiency)	9	8	10	7	8.500
Risk	9	7	8	6	7.500
Experience and technique	7	4	8	7	6.500

* Average=Expert (A+B+C+D)/4

4. The numerical rating of pre-cast concrete pipe lining

Performance indicators	Numerical rating				
	Expert A	Expert B	Expert C	Expert D	Application (Average)
Maintenance stage					
Long term service life(Durability)	8	8	8	8	8.000
Maximum prevention of leakage	8	8	8	8	8.000
Maintenance cost	7	7	8	7	7.250
Quality (Joint, smoothness)	8	8	10	7	8.250
Future plan compliance	8	8	6	7	7.250
Construction stage					
Constructability	6	10	6	6	7.000
Cost and time (Efficiency)	7	5	6	6	6.000
Risk	7	8	6	7	7.000
Experience and technique	8	8	8	6	7.500

* Average=Expert (A+B+C+D)/4

5. The result of evaluation of tunnel lining systems

Performance indicators	Weight	Cast-in-place reinforced concrete		Bolted and gasketed pre-cast segmental concrete		Pre-cast concrete Pipe	
		Rating	Weight * Rating	Rating	Weight * Rating	Rating	Weight * Rating
Maintenance stage	0.625 (A)		5.590 (B)		4.012		4.859
Long term service life (Durability)	0.425 (C)	9.500 (D)	4.038 (E)	6.250	2.656	8.000	3.400
Maximum prevention of leakage	0.175	9.500	1.663 (F)	7.250	1.269	8.000	1.400
Maintenance cost	0.238	8.250	1.959 (G)	5.500	1.306	7.250	1.722
Quality (Joint, smoothness)	0.075	7.500	0.563 (H)	6.500	0.488	8.250	0.619
Future plan Compliance	0.088	8.250	0.722 (I)	8.000	0.700	7.250	0.634
Small sum	1.000	43.000	8.944 (J)	33.500	6.419	38.750	7.775
Construction stage	0.375 (K)		2.524 (L)		2.925		2.527
Constructability	0.225	6.000	1.350 (M)	8.000	1.800	7.000	1.575
Cost and time (Efficiency)	0.338	6.750	2.278 (N)	8.500	2.869	6.000	2.025
Risk	0.288	6.750	1.941 (O)	7.500	2.156	7.000	2.013
Experience and technique	0.150	7.750	1.163 (P)	6.500	0.975	7.500	1.125
Small sum	1.000	27.250	6.731 (Q)	30.500	7.800	27.500	6.738
Total score		8.114 (R)		6.937		7.386	

The evaluation was performed using two types of weighting factors in conjunction with the average of the numerical ratings of each performance indicator according to the following steps:

For example, in the case of cast-in-place reinforced concrete:

- Apply the weighting factor to numerical rating: E= C*D
- Make small sum of different types of tunnel lining systems in each stage:

  J=E+F+G+H+I (in maintenance stage)

  Q=M+N+O+P (in construction stage)
- Weighting factor of each stage is applied to the value of J and Q

  B=A*J

  L=K*Q
- Final value of tunnel lining system

  R=B+L